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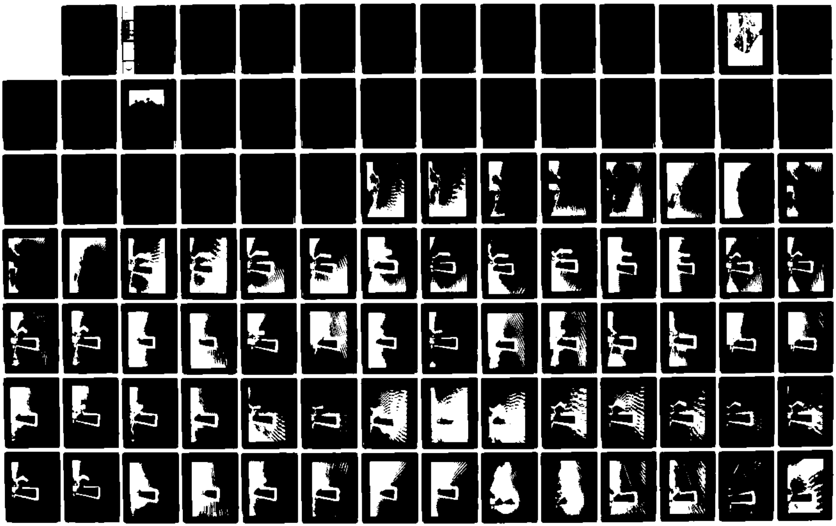

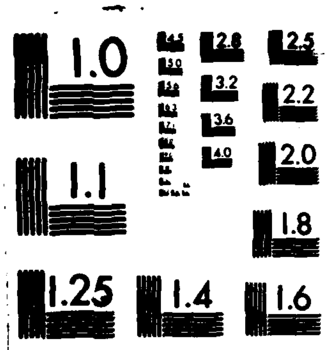

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TECHNICAL REPORT CERC-84-4 (12)

IMPACT OF I-664 BRIDGE/TUNNEL PROJECT ON WAVE CONDITIONS AT NEWPORT NEWS HARBOR, VIRGINIA

Hydraulic Model Investigation

by

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Coastal Engineering Research Center

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Final Report

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A 1:75-scale undistorted hydraulic model was used to investigate the impacts of the proposed Interstate 664 bridge/tunnel project on wave conditions in Newport News Harbor, Virginia. The model included the harbor, approximately 5600 ft of shoreline adjacent to the harbor entrance, and sufficient offshore area in Hampton Roads to permit generation of the required test waves. Various plans of improvement included the use of either rubble-mound or cellular concrete breakwaters. A 50-ft-long wave (Continued)

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20. ABSTRACT (Continued).

generator and an automated data acquisition and control system were utilized in model operation. It was concluded from test results that:

a. Wave conditions in the existing harbor were relatively calm for storm waves from the various test directions. Only the most severe storm waves (50-year recurrence) from 215, 180, and 70 degrees resulted in wave heights in excess of 1.0 ft in the harbor.

b. The most critical direction of wave attack for existing conditions was from 215 degrees since the harbor entrance is more directly exposed to incoming waves from this direction. Wave heights ranging from 1.4 to 2.0 ft may occur for severe storm waves (50-year recurrence) from this direction.

c. The original rubble-mound jetty plan (Plan 1) resulted in wave heights in the harbor well below those obtained for existing conditions. (Maximum wave heights of only 0.9 ft occurred for 50-year storm wave conditions.)

d. The originally proposed rubble-mound jetty can be reduced 300 ft in length to 925 ft (Plan 10) with no adverse effects on wave conditions in the harbor as a result of installation of the north tunnel island and the relocation of the harbor entrance.

e. The crest el of the originally proposed rubble-mound jetty can be reduced by 3 ft in height to +9.3 ft with no adverse effects on wave conditions in the harbor.

f. The original concrete-pile jetty plan (Plan 11) resulted in excessive wave heights (greater than 2 ft) inside the harbor.

g. By sealing the openings between the 54-in. concrete piles (for a distance of 1035 ft) to an el of -4.7 ft (Plan 12), wave heights in the harbor were comparable to those obtained for existing conditions. Plan 12 resulted in no adverse effects on wave conditions in the harbor as a result of the installation of the north tunnel island and relocation of the harbor entrance and was considered the optimum concrete-pile jetty plan tested with respect to wave protection and economics.

h. Even though Plan 12 was considered the optimum concrete-pile jetty in regard to impacts on the harbor, wave heights in the new basin formed by the jetties exceeded 3 ft. By completely sealing the openings between the concrete piles for its entire 2710-ft length (Plan 15) wave heights were reduced to 1.3 ft.

PREFACE

The US Army Engineer District, Norfolk (NAO), initiated a request for the US Army Engineer Waterways Experiment Station (WES) to conduct a model investigation of Newport News Harbor, Va., to study wave conditions as a result of installation of the I-664 Bridge/Tunnel Project proposed by the Virginia Department of Highways and Transportation (VDHT). Authorization for WES to perform the study was granted by the Office, Chief of Engineers, US Army. Funds were authorized by NAO on 30 September 1983, 13 February 1984, 4 April 1984, and 15 June 1984.

The model study was conducted at WES during the period March-June 1984 by personnel of the Wave Processes Branch (WPB), Wave Dynamics Division (WDD), Coastal Engineering Research Center (CERC), under the direction of Dr. R. W. Whalin, Chief of CERC; Dr. L. E. Link, Jr., Assistant Chief of CERC; Mr. C. E. Chatham, Jr., Chief of WDD; and Mr. D. G. Outlaw, Chief of WPB. The tests were conducted by Messrs. H. F. Acuff and M. G. Mize, Civil Engineering Technicians, under the supervision of Mr. R. R. Bottin, Jr., Project Manager. This report was written by Mr. Bottin.

Prior to the model investigation, Messrs. Bottin and Acuff met with representatives of NAO and VDHT and visited Newport News Harbor to inspect the prototype site. During the course of the investigation, liaison between NAO, VDHT, and WES was maintained by means of conferences, telephone communications, and monthly progress reports.

The following personnel visited WES to observe model operation and participate in conferences during the course of the study.

Mr. James Melchor - NAO
Ms. Julie Samuel - NAO
Mr. G. E. Kirk - Federal Highway Administration
Mr. Robert Atherton - VDHT
Mr. Mel Thomas - VDHT
Mr. P. L. Veasey - VDHT
Mr. R. L. Hundley - VDHT

Commander and Director of WES during the conduct of this investigation and preparation and publication of this report was COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, US CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

US customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	25.4	millimetres
miles (US statute)	1.609344	kilometres
miles per hour	1.609344	kilometres per hour
square feet	0.09290304	square metres
square miles	2589.998	square kilometres

IMPACT OF I-664 BRIDGE/TUNNEL PROJECT ON WAVE CONDITIONS AT NEWPORT NEWS HARBOR, VIRGINIA

Hydraulic Model Investigation

PART I: INTRODUCTION

Description of Project

1. The Commonwealth of Virginia Department of Highways and Transportation (VDHT) proposes the construction of a bridge/tunnel crossing at Hampton Roads between the cities of Newport News and Suffolk (VDHT 1978). Hampton Roads is a body of water located in the southeastern corner of the state (adjacent to the Chesapeake Bay) into which the James, Elizabeth, and Nansemond Rivers empty. Figure 1 shows the project location as well as the proposed bridge/tunnel project.

2. The new Hampton Roads crossing (designated I-664 bridge/tunnel) has been under consideration and discussion for many years. It will complete the ongoing construction of the 13.1-mile* Interstate Route 664 project and help meet the projected 1995 traffic demands between two major metropolitan regions. A highway bridge is proposed for construction across the southern portion of Hampton Roads and a tunnel will pass below the existing 45-ft-deep Newport News navigation channel at the northern portion of Hampton Roads. Islands are needed at each end of the I-664 tunnel to provide a transition between the underwater tunnel and the surface approach structures.

Modifications at Small-Boat Harbor Entrance

3. Due to the restrictions imposed by the site of the Hampton Roads Sanitation District treatment facility (located west of the existing Newport News harbor entrance), the proposed highway alignment will cross the existing small-boat harbor entrance. The construction of the north tunnel island at this location will require reconstruction of the harbor entrance and

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* A table of factors for converting US customary units of measurements to metric (SI) units is presented on page 3.

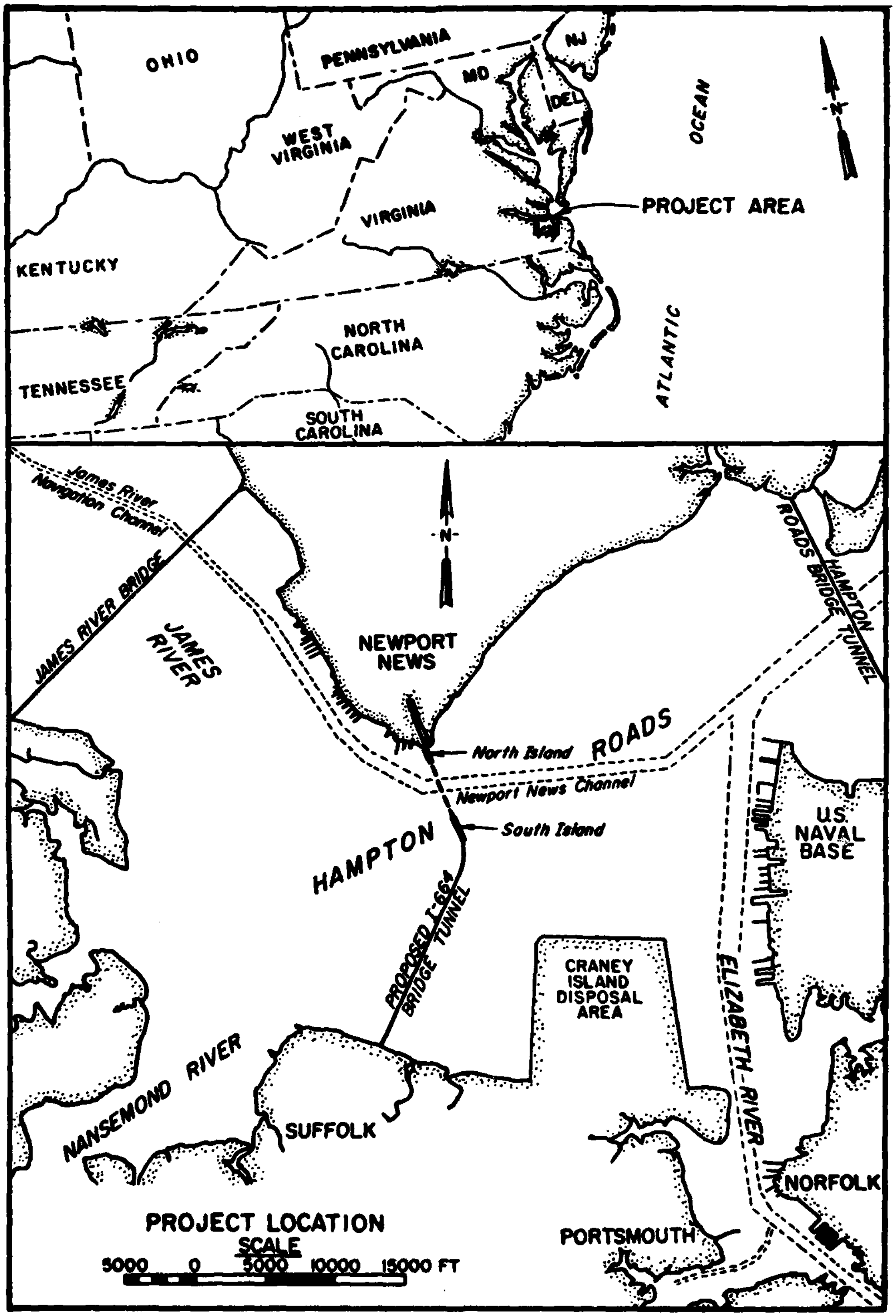

Figure 1. Project location

relocation of the existing Federal Project Channel (FPC). An aerial photograph of the existing small-boat harbor is shown in Figure 2. The VDHT proposes to relocate the harbor entrance about 150 ft eastward of its present location, and provide an increase in the FPC entrance width from 60 to 90 ft to accommodate future improvements in the area. From the new harbor entrance, the relocated FPC would extend bayward with a 150-ft-wide channel bounded on the west by the proposed 1,400-ft-long north tunnel island and on the east by a jetty.

4. The VDHT proposes the construction of a rubble-mound jetty east of the relocated FPC to provide wave protection from storm-generated waves for vessels entering and/or moored in the harbor. This jetty also would prevent shoaling in the entrance. An alternate jetty design, consisting of concrete cylinder piles, also is under consideration in response to requests by the city of Newport News. This jetty should provide the same level of protection as the rubble-mound design while providing an area for additional harbor space for the city's future development.

Purpose of the Model Investigation

5. At the request of the US Army Engineer District, Norfolk (NAO), a hydraulic model investigation was conducted by the US Army Engineer Waterways Experiment Station (WES) to:

- a. Determine wave conditions in the existing small-boat harbor.
- b. Determine wave conditions in the harbor as a result of the proposed modifications to the harbor entrance.
- c. Develop remedial plans, as necessary, for the alleviation of undesirable wave conditions.
- d. Determine if suitable design modifications of the jetties can be made that would reduce construction costs without sacrificing adequate wave protection.

6. Tests involving the impact of the proposed I-664 bridge/tunnel complex on sedimentation and tidal circulation were performed in the Hydraulics Laboratory at WES and are reported by Heltzel (1984).

Figure 2. Aerial view of the existing small-boat harbor

PART II: THE MODEL

Design of Model

7. The Newport News Harbor model (Figure 3) was constructed to an undistorted linear scale of 1:75, model to prototype. Scale selection was based on such factors as:

- a. Depth of water required in the model to prevent excessive bottom friction.
- b. Absolute size of model waves.
- c. Available shelter dimensions and area required for model construction.
- d. Efficiency of model operation.
- e. Available wave-generating and wave-measuring equipment.
- f. Model construction costs.

A geometrically undistorted model was necessary to ensure accurate reproduction of short-period wave and current patterns. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law (Stevens et al. 1942). The scale relations used for design and operation of the model were as follows:

Characteristic	Dimension*	Model:Prototype Scale Relation
Length	L	$L_r = 1:75$
Area	L^2	$A_r = L_r^2 = 1:5,625$
Volume	L^3	$V_r = L_r^3 = 1:421,875$
Time	T	$T_r = L_r^{1/2} = 1:8.66$
Velocity	L/T	$V_r = L_r^{1/2} = 1:8.66$

* Dimensions are in terms of length and time.

8. The proposed improvement plans tested in the Newport News model included the use of rubble-mound revetments adjacent to the bridge/tunnel island, and some plans entailed a rubble-mound jetty. Also, the existing revetments and absorbers are rubble-mound structures. Experience and experimental research have shown that considerable wave energy passes through the interstices of this

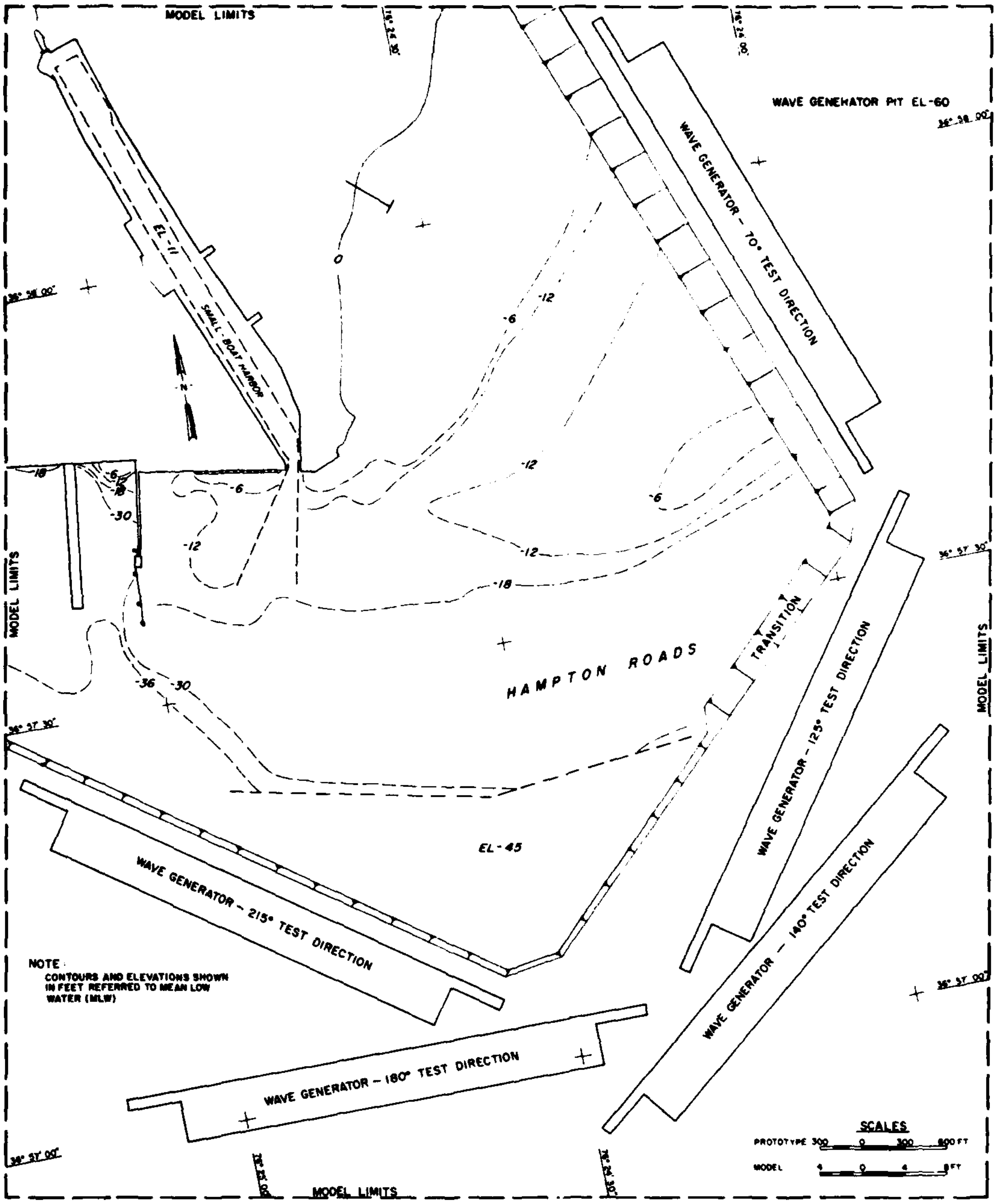

Figure 3. Model layout

type of structure; thus, the transmission and absorption of wave energy became a matter of concern in design of the 1:75-scale model. In small-scale hydraulic models, rubble-mound structures reflect relatively more and absorb or dissipate relatively less wave energy than geometrically similar prototype structures (LeMéhauté 1965). Also the transmission of wave energy through a rubble-mound jetty is relatively less for the small-scale model than for the prototype. Consequently, some adjustment in small-scale model rubble-mound structures is needed to ensure satisfactory reproduction of wave-reflection and wave-transmission characteristics. In past investigations at WES (Dai and Jackson 1966, Brasfeild and Ball 1967), this adjustment was made by determining the wave-energy transmission characteristics of the proposed structure in a two-dimensional model using a scale large enough to ensure negligible scale effects. A breakwater section then was developed for the small-scale, three-dimensional model that would provide essentially the same relative transmission of wave energy. Therefore, from previous findings for rubble-mound structures and wave conditions similar to those at Newport News, it was determined that a close approximation of the correct wave-energy transmission characteristics would be obtained by increasing the size of the rock used in the 1:75-scale model to approximately one and a half times that required for geometric similarity. Accordingly, in constructing the breakwater structures in the Newport News Harbor model, the rock sizes were computed by linear scale, then multiplied by 1.5 to determine the actual sizes to be used in the model.

The Model and Appurtenances

9. The model, which was molded in cement mortar, reproduced the entire Newport News Harbor, approximately 3,600 and 2,000 ft of shoreline to the east and west of the harbor entrance, respectively, and underwater contours in Hampton Roads to an offshore depth of -36 ft (including a portion of the -45 ft Newport News Channel), with a sloping transition to the wave generator pit elevation (el) of -60 ft.* The total area reproduced in the model was approximately 11,050 sq ft, representing about 2.2 square miles in the prototype. A general view of the model is shown in Figure 4. Vertical control for model construction was based on mean low water (mlw). Horizontal control was referenced to a local prototype grid system.

10. Model waves were generated by a 50-ft-long wave generator with a trapezoidal-shaped, vertical-motion plunger. The vertical movement of the plunger caused a periodic displacement of water incident to this motion. The length of the stroke and the frequency of the vertical motion were variable over the range necessary to generate waves with the required characteristics.

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* All elevations (el) cited herein are in feet referred to mean low water.

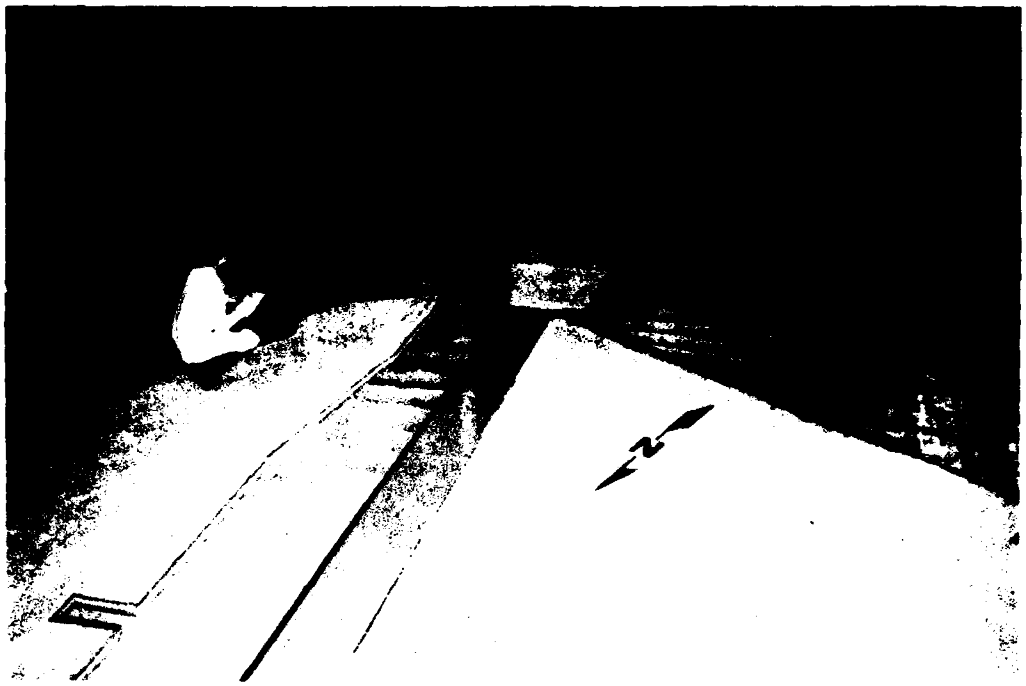

Figure 4. General view of model

In addition, the wave generator was mounted on retractable casters which enabled it to be positioned to generate waves from the required directions.

11. An Automated Data Acquisition and Control System (ADACS), designed and constructed at WES (Figure 5), was used to secure wave-height data at selected locations in the model. Basically, through the use of a minicomputer, ADACS recorded onto magnetic tape the electrical output of parallel-wire, resistance-type sensors which measured the change in water-surface elevation with respect to time. The magnetic tape output then was analyzed to obtain the required data.

12. Guide vanes were placed along the wave generator sides to ensure proper formation of the wave train incident to the model contours. In addition, a 2-ft (horizontal) solid layer of fiber wave absorber was placed around the inside perimeter of the model to dampen any wave energy that might otherwise be reflected from the model walls.

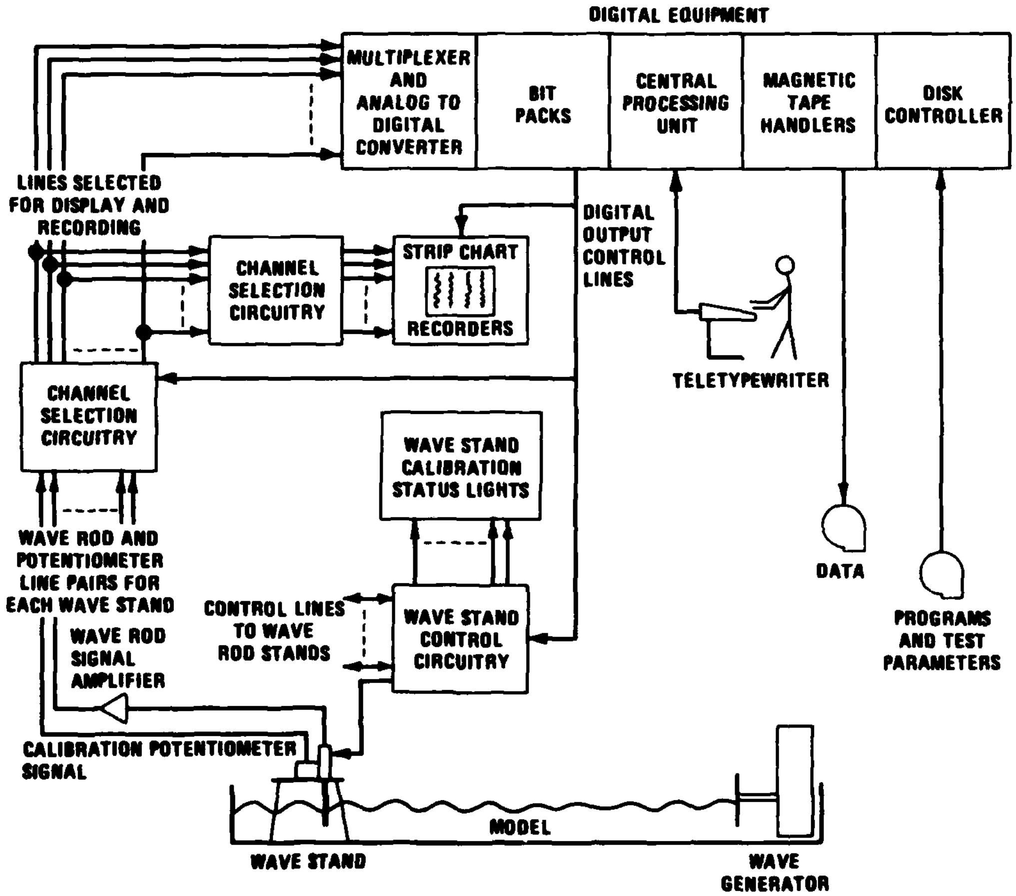

Figure 5. Automated Data Acquisition and Control System (ADACS)

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

Still water level

13. Still water levels (swl) for harbor wave action models are selected so that the various wave-induced phenomena that are dependent on water depths are accurately reproduced in the model. These phenomena include the refraction of waves in the harbor area, the overtopping of harbor structures by the waves, the reflection of wave energy from harbor structures, and the transmission of wave energy through porous structures. In most cases it is desirable to select a model swl that closely approximates the higher water stages which normally occur in the prototype for the following reasons:

- a. The maximum amount of wave energy reaching a coastal area normally occurs during the higher water phase of the local tidal cycle.
- b. Most storms moving onshore are characteristically accompanied by a higher water level due to wind tide and shoreward mass transport.
- c. The selection of a high swl helps minimize model scale effects due to viscous bottom friction.
- d. When a high swl is selected, a model investigation tends to yield more conservative results.

14. A swl of +2.6 ft was selected by NAO for use during model testing. This value represents mean high water (mhw).

Factors influencing selection of test wave characteristics

15. In planning the testing program for a model investigation of harbor wave-action problems, it is necessary to select dimensions and directions for the test waves that will allow a realistic test of proposed improvement plans and an accurate evaluation of the elements of the various proposals. Surface-wind waves are generated primarily by the interactions between tangential stresses of wind flowing over water, resonance between the water surface and atmospheric turbulence, and interactions between individual wave components. The height and period of the maximum wave that can be generated by a given storm depend on the wind speed, the length of time that wind of a given speed continues to blow, and the water distance (fetch) over which the wind blows. Selection of test wave conditions entails evaluation of such factors as:

a. The fetch and decay distances (the latter being the distance over which waves travel after leaving the generation area) for various directions from which waves can attack the problem area.

b. The frequency of occurrence and duration of storm winds from the different directions.

c. The alignment, size, and relative geographic position of the navigation entrance to the harbor.

d. The alignments, lengths, and locations of the various reflecting surfaces inside the harbor.

e. The refraction of waves caused by differentials in depth in the area bayward of the harbor, which may create either a concentration or a diffusion of wave energy at the harbor site.

Wave refraction

16. When wind waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period (to the first order of approximation). The most important transformations with respect to the selection of test wave characteristics are the changes due to wave refraction and shoaling. The change in wave height and direction can be determined by conducting a wave refraction analysis. The shoaling coefficient, a function of wave length and water depth, can be obtained from US Army CERC (1977). Thus, the refraction coefficient multiplied by the shoaling coefficient gives a conversion factor for transfer of deepwater wave heights to shallow-water values.

17. Due to the limited fetch in Hampton Roads, a wave refraction analysis was not conducted for the Newport News Harbor site. The magnitude and direction of winds approaching the area from over the Hampton Roads water body were considered to be the governing factors and all waves were assumed to be locally generated. For this study, critical directions of wave approach were determined to be from 215, 180, 140, 125, and 70 deg.

Selection of test waves

18. Measured prototype wave data on which a comprehensive statistical analysis of wave conditions could be based were unavailable for the Newport News area. However, statistical wave hindcast data representative of this area were obtained by the application of hindcasting techniques from Vincent and Lockhart (1983) to wind data acquired from Resio et al. (1982) and Brooks and Corson (1984). Statistical distribution of these wind data was obtained using methods employed in Bowers et al. (1971). The following tabulation shows selected test directions, fetch lengths, depths associated with these fetches,

1- and 50-year wind speeds, and corresponding test wave characteristics which were utilized in model operation.

Direction deg	Fetch ft	Depth ft	Wind speed, mph 1 year	50 year	Period sec	Height ft
215	36,500	17	31		3.2*	2.5
				53	3.9	4.5
180	22,500	20	30		2.8*	2.3
				52	3.6	4.1
140	17,800	25	26		2.5*	1.7
				53	3.3*	3.7
125	30,000	26	28		3.0*	2.4
				45	3.6	3.8
70	134,900	36	26		4.3	4.0
				60	6.0	9.0

* Due to limitations of the model wave generator, it was necessary to select wave periods of 3.5 sec and above. Therefore, the 2.5-, 2.8-, 3.0-, 3.2-, and 3.3-sec wave periods were not generated in the model but replaced with 3.5-sec periods.

Analysis of Model Data

19. Relative merits of the various plans tested were evaluated by:

a. Comparison of wave heights at selected locations in the model.

b. Visual observations and wave pattern photographs.

In the wave-height data analysis, the average height of the highest one-third of the waves recorded at each gage location ($H_{1/3}$) was computed. All wave heights then were adjusted to compensate for excessive model wave height attenuation due to viscous bottom friction by application of Keulegan's equation (Keulegan 1950). From this equation reduction of wave heights in the model (relative to the prototype) can be calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel.

PART IV: TESTS AND RESULTS

The Tests

Existing conditions

20. Prior to testing of the various improvement plans, tests were conducted for existing conditions (Plate 1). Wave heights and wave pattern photographs were obtained for test waves from the five test directions.

Improvement plans

21. Wave height tests were conducted for 18 variations in the design elements of two basic jetty plans. These variations consisted of changes in the lengths, alignments, locations, and/or cross sections of the jetties. Wave pattern photographs were obtained for all the test plans. Brief descriptions of the improvement plans are presented in the following subparagraphs; dimensional details are presented in Plates 2-17.

- a. Plan 1 (Plate 2) consisted of the construction of the north tunnel island bayward of the existing entrance to the harbor and the relocation of the harbor entrance about 150 ft eastward. A new 14-ft-deep, 150-ft-wide entrance channel extended bayward adjacent to and east of the tunnel island. A 1,225-ft-long rubble-mound jetty (el +12.3 ft) was installed east of the entrance channel.
- b. Plan 2 (Plate 3) entailed the elements of Plan 1 but the rubble-mound jetty (el +12.3 ft) was decreased to 850 ft in length.
- c. Plan 3 (Plate 4) involved the elements of Plan 1 but the rubble-mound jetty (el +12.3 ft) was decreased to 700 ft in length.
- d. Plan 4 (Plate 5) included the elements of Plan 1 but the rubble-mound jetty (el +12.3 ft) was decreased to 600 ft in length.
- e. Plan 5 (Plate 6) encompassed the elements of Plan 1 but the rubble-mound jetty (el +12.3 ft) was decreased to 300 ft in length.
- f. Plan 6 (Plate 7) entailed the 600-ft-long rubble-mound jetty of Plan 4 but the crest el was reduced to +9.3 ft.
- g. Plan 7 (Plate 8) included the 700-ft-long rubble-mound jetty of Plan 3 but the crest el was reduced to +9.3 ft.
- h. Plan 8 (Plate 9) involved the 850-ft-long rubble-mound jetty of Plan 2 but the crest el was reduced to +9.3 ft.
- i. Plan 9 (Plate 10) encompassed the elements of Plan 1 with a 1,000-ft-long rubble-mound jetty (el +9.3 ft).
- j. Plan 10 (Plate 11) involved the elements of Plan 1 with a 925-ft-long rubble-mound jetty (el +9.3 ft).

k. Plan 11 (Plate 12) consisted of a 2,710-ft-long concrete cylinder-pile breakwater (el +9.3 ft). The piles were spaced 6 in. apart and the openings between the piles were sealed with timber to an el of -0.7 ft. The structure originated from shore east of the harbor entrance and extended lakeward 1,675 ft (66-in.-diam concrete piles). From this point the structure extended 1,035 ft toward the tunnel island (54-in.-diam concrete piles).

l. Plan 12 (Plate 13) involved the elements of Plan 11 but the openings between the 54-in.-diam piles were sealed to an el of -4.7 ft.

m. Plan 13 (Plate 14) entailed the elements of Plan 11 but the openings between both the 54- and 66-in.-diam piles were sealed to an el of -4.7 ft.

n. Plan 14 (Plate 15) included the elements of Plan 11 but the openings between the 54-in.-diam piles were sealed to -4.7 ft and the openings between the 66-in.-diam piles were completely sealed to the bay bottom.

o. Plan 15 (Plate 16) encompassed the elements of Plan 11 but the openings between both the 54- and 66-in.-diam piles were completely sealed to the bay bottom.

p. Plan 16 (Plate 17) involved the elements of Plan 11 but the openings between the 54-in.-diam piles were completely sealed to the bay bottom.

q. Plan 17 (Plate 17) entailed the elements of Plan 11 but the openings between the 54-in.-diam piles were completely sealed to the bay bottom at the eastern 525-ft portion of the structure.

r. Plan 18 (Plate 17) included the elements of Plan 11 but the openings between the 54-in.-diam piles were completely sealed to the bay bottom at the eastern 790-ft portion of the structure.

Wave height tests

22. Wave height tests for the various improvement plans were conducted using test waves from one or more of the directions listed in Paragraph 18. Tests involving certain proposed improvement plans were limited to the most critical direction of wave approach (i.e. 140, 125, and/or 70 deg). The most promising plans of improvement (Plans 10 and 12) were tested comprehensively for waves from all five test directions. Wave gage locations for each improvement plan are shown in Plates 2-17.

Video tape

23. Video tape footage of the Newport News Harbor model was secured for existing conditions and Plans 1, 10, and 12 showing the area under attack by storm waves approaching from the 140-deg test direction. This footage was

furnished to NAO for use in briefings, public meetings, etc.

Test Results

24. In evaluating test results, the relative merits of various plans were based on an analysis of measured wave heights in the mooring area and entrance. Model wave heights (significant wave height or $H_{1/3}$) were tabulated to show measured values at selected locations.

Existing conditions

25. Wave height measurements obtained for existing conditions are presented in Table 1. Maximum wave heights obtained were 4.7 ft in the entrance (gage 6) and 2.0 ft inside the harbor (gage 5) for 3.9-sec, 4.5-ft test waves from 215 deg. The present harbor entrance is more directly exposed to waves from 215 deg than it is to waves from the other test directions. Typical wave patterns for existing conditions are shown in Photos 1-10.

Improvement plans

26. Results of wave height tests conducted for the original rubble-mound test plan (Plan 1) for test waves from the various directions are presented in Table 2. Maximum wave heights were 6.9 ft in the area between the jetty head and the north tunnel island (gage 9) for 6-sec, 9-ft test waves from 70 deg; 0.8 ft in the harbor entrance (gage 6) for 6-sec, 9-ft test waves from 70 deg, and 0.8 ft also for 3.5-sec, 3.7-ft test waves from 140 deg; and 0.9 ft inside the harbor (gage 5) for 6-sec, 9-ft test waves from 70 deg. Wave pattern photographs obtained for Plan 1 are shown in Photos 11-20.

27. Wave heights obtained for Plans 2-6 for test waves from 140 and 125 deg are presented in Table 3. For test waves from 140 deg, maximum wave heights were 1.9, 2.0, 2.0, 2.3, and 2.2 ft in the harbor entrance (gage 6) and 0.8, 1.0, 1.0, 1.2, and 1.1 ft inside the harbor for Plans 2-6, respectively. With test waves from 125 deg, maximum wave heights in the harbor entrance were 1.4, 1.6, 2.0, 2.1, and 2.2 ft and maximum wave heights inside the harbor were 0.7, 0.9, 1.0, 1.6, and 1.2 ft for Plans 2-6, respectively. Typical wave patterns for Plans 2-6 for test waves from 140 and 125 deg are shown in Photos 21-40.

28. Results of wave height tests for Plans 6-10 are presented in Table 4 for test waves from 70 deg. Maximum wave heights obtained were 3.5, 2.8, 2.1, 0.9, and 1.3 ft in the harbor entrance and 4.3, 3.0, 2.4, 1.2, and

1.6 ft inside the harbor for Plans 6-10, respectively. Wave pattern photographs obtained for Plans 6-10 are shown in Photos 41-50 for test waves from 70 deg.

29. Wave heights secured for Plan 10 for test waves from 125, 140, 180, and 215 deg are presented in Table 5. Maximum wave heights were 3.7 ft in the area between the jetty head and the north tunnel island (gage 8); 2.0 ft in the harbor entrance (gage 6); and 1.2 ft inside the harbor (gage 5) all for 3.5-sec, 3.7-ft test waves from 140 deg. Typical wave patterns for Plan 10 are shown in Photos 51-58 for test waves from 125, 140, 180, and 215 deg.

30. Wave heights obtained for Plan 11 for test waves from 140, 125, and 70 deg are presented in Table 6. Maximum wave heights were 5.5 ft in the area between the jetty head and the north tunnel island (gage 9); 2.7 ft in the harbor entrance (gage 6); 2.3 ft inside the harbor (gage 5); and 5.2 ft inside the new basin formed by the jetties (gage 10), all for 6-sec, 9-ft test waves from 70 deg. Typical wave patterns secured for Plan 11 for test waves from 140, 125, and 70 deg are shown in Photos 59-64.

31. Wave height data secured for Plans 12-18 for test waves from 70 deg are presented in Table 7. Maximum wave heights obtained were 1.7, 1.7, 1.2, 0.4, 0.8, 1.9, and 0.9 ft in the harbor entrance; 1.4, 1.5, 1.3, 0.5, 0.6, 2.1, and 0.9 ft inside the harbor; and 3.0, 2.9, 2.9, 1.3, 2.6, 3.3, and 2.7 ft inside the new basin formed by the jetties for Plans 12-18, respectively. Wave patterns obtained for Plans 12-18 are shown in Photos 65-78 for test waves from 70 deg.

32. Results of wave height tests for Plan 12 for test waves from 125, 140, 180, and 215 deg are presented in Table 8. Maximum wave heights were 3.9 ft in the area between the jetty head and the north tunnel island (gage 9) for 3.6-sec, 4.1-ft test waves from 180 deg; 1.2 ft in the harbor entrance (gage 6) for 3.5-sec, 3.7-ft test waves from 140 deg; 0.7 ft in the harbor (gage 5) for 3.5-sec, 3.7-ft test waves from 140 deg; and 2.2 ft in the new basin formed by the jetties (gage 10) for 3.6-sec, 4.1-ft test waves from 180 deg. Typical wave patterns obtained for Plan 12 are shown in Photos 79-86 for test waves from 125, 140, 180, and 215 deg.

Discussion of test results

33. Results of wave height tests for existing conditions indicated that the 215-deg test direction was the most critical with regard to wave conditions in the entrance and inside the harbor since the entrance is more directly

exposed from this direction. For the most severe storm waves from this direction (50-year recurrence interval), wave heights inside the harbor will range from 1.4 to 2.0 ft. Waves with 50-year recurrence intervals from 70 deg will result in 1.5-ft wave heights in the harbor, and for 50-year waves from 180 deg, 1.2-ft wave heights will be experienced inside the harbor. For storm waves with 1-year recurrence intervals, however, a maximum wave height of only 1.1 ft was obtained inside the harbor from 215 deg. Wave heights inside the harbor for test waves for 70-180 deg were less than 0.5 ft for 1-year storm conditions.

34. Wave height test results obtained for the initial rubble-mound jetty plan (Plan 1) revealed no problems in the harbor entrance or inside the harbor. Maximum wave heights inside the harbor for 1-year storm conditions were only 0.2 ft and maximum wave heights inside the harbor for 50-year storm conditions were 0.9 ft. These tests resulted in wave heights in the harbor well below those obtained for existing conditions and indicated that the structure length may be reduced without any adverse effects to the harbor.

35. Since the entrance of the proposed improvement plan was oriented toward waves from 140-125 deg, the initial rubble-mound jetty was incrementally reduced in length (Plans 2-5) and subjected to test waves from 140 and 125 deg. For waves from these directions, it appeared that the 600-ft-long jetty of Plan 4 was optimum in regard to wave heights obtained in the harbor and construction costs of the jetty.

36. The decrease in the height of the Plan 4 jetty crest el (+12.3 ft) to +9.3 ft (Plan 6) resulted in a negligible change in wave heights in the harbor; therefore, the Plan 6 jetty length and crest height were considered optimum at that point for test waves from 140 and 125 deg.

37. Additional testing of the Plan 6 jetty for waves from 70 deg indicated wave heights in the harbor in excess of 4 ft for 50-year storm wave conditions. Visual observations revealed wave energy diffracting around the head of the jetty into the harbor entrance. The length of the jetty was incrementally increased (Plans 7-10) and subjected to test waves from 70 deg. Evaluation of these data indicated that a 925-ft-long jetty (Plan 10) was optimum considering wave heights in the harbor and construction costs. Maximum wave heights in the harbor were 1.6 ft as opposed to 1.5-ft wave heights for existing conditions for 50-year storm waves. Wave height tests conducted for waves from 125-215 deg revealed that only 50-year storm conditions from 140 deg

would result in wave heights in excess of 1.0 ft inside the harbor (1.2 ft).

38. In summary, test results for the various rubble-mound jetty plans (Plans 1-10) indicated that a 925-ft-long jetty with a crest el of +9.3 ft (Plan 10) was optimum and would alleviate undesirable wave conditions in the harbor as a result of the installation of the north tunnel island and relocation of the harbor entrance. The originally proposed structure (Plan 1) length was reduced by 300 ft and the crest el was reduced by 3 ft as a result of the model investigation.

39. Wave height test results obtained for the initial concrete-pile breakwater plan (Plan 11) revealed wave heights in the harbor entrance of 2.7 ft and wave heights inside the harbor of 2.3 ft for 50-year test waves from 70 deg. Wave conditions in the new basin formed by the jetties were in excess of 5 ft in some locations. Visual observations revealed wave energy entering the area through the 6-in. openings between the piles (particularly the 54-in. pile structures).

40. By sealing the openings between the 54-in. concrete-pile jetties to -4.7 ft (Plan 12), wave heights in the harbor were reduced to 1.4 ft for test waves from 70 deg. This was comparable to heights obtained for existing conditions (1.5 ft in the harbor). Wave heights in the new area formed by the jetties were in excess of 3 ft, however. While not within the original scope of this study, additional plans were tested in an effort to reduce wave heights in this area. Of the plans tested, Plan 15 (openings in entire structure completely sealed) resulted in wave heights of 1.3 ft for 50-year wave conditions. Visual observations indicated energy entering the harbor due to diffracted wave patterns at the entrance.

41. In summary, test results for the various concrete-pile jetty plans (Plans 11-18) indicated that the 6-in. openings between the concrete piles had to be sealed to an el of -4.7 ft (Plan 12) on the bayward side. This plan was optimum considering wave impacts on the harbor as a result of the installation of the north tunnel island and relocation of the harbor entrance; however, wave heights in the new basin formed by the jetties exceeded 3 ft. Wave heights in this area could be reduced to 1.3 ft by completely sealing the openings for the entire 2,710-ft length of the structure.

PART V: CONCLUSIONS

42. Based on the results of the hydraulic model investigation reported herein, it was concluded that

a. Wave conditions in the existing harbor were relatively calm for storm waves from the various test directions. Only the most severe storm waves (50-year recurrence) from 215, 180, and 70 deg resulted in wave heights in excess of 1.0 ft in the harbor.

b. The most critical direction of wave attack for existing conditions was from 215 deg since the harbor entrance is more directly exposed to incoming waves from this direction. Wave heights ranging from 1.4 to 2.0 ft may occur for severe storm waves (50-year recurrence) from this direction.

c. The original rubble-mound jetty plan (Plan 1) resulted in wave heights in the harbor well below those obtained for existing conditions. (Maximum wave heights of only 0.9 ft occurred for 50-year storm wave conditions).

d. The originally proposed rubble-mound jetty can be reduced 300 ft in length to 925 ft (Plan 10) with no adverse effects on wave conditions in the harbor as a result of the installation of the north tunnel island and the relocation of the harbor entrance.

e. The crest el of the originally proposed rubble-mound jetty can be reduced by 3 ft in height to +9.3 ft with no adverse effects on wave conditions in the harbor.

f. The original concrete-pile jetty plan (Plan 11) resulted in excessive wave heights (greater than 2 ft) inside the harbor.

g. By sealing the openings between the 54-in. concrete piles (for a distance of 1035 ft) to an el of -4.7 ft (Plan 12), wave heights in the harbor were comparable to those obtained for existing conditions. Plan 12 resulted in no adverse effects on wave conditions in the harbor as a result of the installation of the north tunnel island and relocation of the harbor entrance and was considered the optimum concrete-pile jetty plan tested with respect to wave protection and economics.

h. Even though Plan 12 was considered the optimum concrete-pile jetty in regard to impacts on the harbor, wave heights in the new basin formed by the jetties exceeded 3 ft. By completely sealing the openings between the concrete piles for its entire 2710-ft length (Plan 15) wave heights were reduced to 1.3 ft.

Table 1

Wave Heights for Existing Conditions

Direction deg	Test Wave Period sec	Test Wave Height ft	Wave Height, ft, at Indicated Gage Locations 1	2	3	4	5	6	7
70	4.3	4.0	0.3	0.2	0.4	0.3	0.3	1.2	2.2
	6.0	9.0	0.9	1.1	1.5	0.5	0.8	2.6	3.7
125	3.5	2.4	<0.1	<0.1	0.1	0.2	0.1	0.8	1.7
	3.6	3.8	0.2	0.3	0.6	0.8	0.6	2.1	4.5
140	3.5	1.7	<0.1	0.1	0.1	0.3	0.1	0.9	2.2
	3.5	3.7	0.2	0.3	0.3	0.6	0.4	1.9	3.3
180	3.5	2.3	0.1	<0.1	0.2	0.5	0.2	1.4	2.4
	3.6	4.1	0.2	0.3	0.5	1.2	0.7	2.5	4.4
215	3.5	2.5	0.3	0.3	1.0	0.5	1.1	2.7	3.0
	3.9	4.5	1.4	1.7	1.6	1.7	2.0	4.7	3.3

Table 2

Wave Heights for Plan 1

Direction deg	Test Wave Period sec	Test Wave Height ft	Wave Height, ft, at Indicated Gage Locations 1	2	3	4	5	6	7	8	9
70	4.3	4.0	0.1	0.2	<0.1	<0.1	<0.1	0.2	0.1	0.5	2.2
	6.0	9.0	0.3	0.4	0.5	0.5	0.9	0.8	1.6	2.4	6.9
125	3.5	2.4	<0.1	<0.1	<0.1	0.1	0.1	0.2	0.1	0.4	2.1
	3.6	3.8	<0.1	0.1	<0.1	0.2	0.3	0.4	0.4	1.1	4.0
140	3.5	1.7	<0.1	<0.1	<0.1	<0.1	<0.1	0.3	0.3	0.5	0.9
	3.5	3.7	0.1	0.2	0.2	0.4	0.4	0.8	2.1	2.3	4.0
180	3.5	2.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.2	0.3	0.9
	3.6	4.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.1	0.7	0.9	2.4
215	3.5	2.5	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.1	0.1	0.6
	3.9	4.5	<0.1	<0.1	<0.1	<0.1	0.1	0.2	0.2	0.4	0.5

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Table 3

Wave Heights for Plans 2-6 for Test Waves from 140 and 125 Deg

	Test Wave		Wave Height, ft, at Indicated Gage Locations								
Plan	Period sec	Height ft	1	2	3	4	5	6	7	8	9
						140 deg					
2	3.5	1.7	<0.1	<0.1	<0.1	0.1	0.2	0.4	0.2	1.1	1.3
	3.5	3.7	0.2	0.3	0.3	0.7	0.8	1.9	2.2	3.6	4.2
3	3.5	1.7	0.1	<0.1	<0.1	0.4	0.8	0.8	1.0	1.1	1.2
	3.5	3.7	0.2	0.3	0.3	0.7	1.0	2.0	2.1	3.3	4.3
4	3.5	1.7	0.2	0.2	<0.1	0.4	0.6	1.3	0.7	1.2	1.4
	3.5	3.7	0.3	0.4	0.3	0.8	1.0	2.0	2.0	3.3	4.5
5	3.5	1.7	0.2	0.3	<0.1	0.6	0.7	1.5	0.9	1.2	1.2
	3.5	3.7	0.3	0.6	0.3	0.9	1.2	2.3	3.0	4.5	4.2
6	3.5	1.7	0.2	0.2	<0.1	0.5	0.5	1.2	0.9	1.2	1.6
	3.5	3.7	0.3	0.5	0.3	0.8	1.1	2.2	2.0	3.6	4.3
						125 deg					
2	3.5	2.4	<0.1	0.1	<0.1	0.3	0.3	0.4	0.7	2.4	1.5
	3.6	3.8	0.3	0.4	0.3	0.6	0.7	1.4	1.2	3.5	4.7
3	3.5	2.4	<0.1	0.1	<0.1	0.4	0.6	0.9	0.5	2.1	1.5
	3.6	3.8	0.3	0.4	0.4	0.8	0.9	1.6	2.3	3.4	3.8
4	3.5	2.4	0.2	0.3	<0.1	0.7	0.9	1.1	1.5	2.1	1.8
	3.6	3.8	0.4	0.5	0.5	0.9	1.0	2.0	2.6	3.7	4.3
5	3.5	2.4	0.3	0.4	0.3	0.6	0.9	1.6	1.6	2.4	1.8
	3.6	3.8	0.4	0.6	1.3	1.6	0.8	2.1	4.4	2.4	3.6
6	3.5	2.4	0.3	0.3	0.1	0.6	0.8	1.2	1.4	1.6	2.2
	3.6	3.8	0.4	0.4	0.5	0.8	1.2	2.2	3.1	3.6	4.1

Table 4

Wave Heights for Plans 6-10 for Test Waves from 70 Deg

Plan	Test Wave Period sec	Test Wave Height ft	Wave Height, ft, at Indicated Gage Locations 1	2	3	4	5	6	7	8	9
6	4.3	4.0	0.6	0.7	0.4	0.8	0.7	1.0	1.0	3.1	2.3
	6.0	9.0	0.8	1.7	1.6	3.4	4.3	3.5	3.5	6.0	6.9
7	4.3	4.0	0.4	0.5	0.3	0.3	0.2	0.6	0.4	3.1	4.6
	6.0	9.0	0.9	1.5	1.2	2.8	3.0	2.8	2.8	4.1	6.3
8	4.3	4.0	0.4	0.3	0.2	0.4	0.1	0.6	0.2	2.4	4.5
	6.0	9.0	1.1	1.2	0.7	2.4	2.2	2.1	2.4	3.7	6.2
9	4.3	4.0	0.2	0.2	0.1	0.3	0.2	0.6	1.2	3.2	2.7
	6.0	9.0	0.5	0.7	0.4	1.2	1.1	0.9	1.9	5.6	6.4
10	4.3	4.0	0.2	0.3	0.3	0.4	0.4	0.8	1.1	3.0	3.9
	6.0	9.0	0.5	1.1	0.5	1.6	1.3	1.3	1.7	6.2	5.1

Table 5

Wave Heights for Plan 10 for Test Waves from 125-215 Deg

Direction deg	Test Wave Period sec	Test Wave Height ft	Wave Height, ft, at Indicated Gage Locations 1	2	3	4	5	6	7	8	9
125	3.5	2.4	0.1	0.1	0.1	0.3	0.2	0.5	0.4	1.4	1.7
	3.6	3.8	0.3	0.6	0.4	0.7	0.8	1.7	1.7	3.4	4.3
140	3.5	1.7	0.2	0.2	0.3	0.7	0.6	0.9	0.9	1.2	1.9
	3.5	3.7	0.4	0.5	0.6	1.0	1.2	2.0	2.5	3.7	4.2
180	3.5	2.3	<0.1	<0.1	<0.1	<0.1	<0.1	0.1	0.3	0.7	1.5
	3.6	4.1	<0.1	0.1	<0.1	0.2	0.2	0.4	0.9	1.6	2.9
215	3.5	2.5	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.3	0.7
	3.9	4.5	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.1	0.3	0.8

Table 8

Wave Heights for Plan 12 for Test Waves from 125-215 Deg

Direction deg	Test Wave Period sec	Test Wave Height ft	Wave Height, ft, at Indicated Gage Locations 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
125	3.5	2.4	<0.1	<0.1	<0.1	0.1	0.2	0.2	0.4	1.2	2.0	0.5	0.5	0.4	0.9	0.4	0.4
	3.6	3.8	0.1	0.2	0.2	0.3	0.3	0.7	1.1	2.2	3.5	0.6	0.7	0.9	1.6	1.5	0.9
140	3.5	1.7	0.1	0.2	0.2	0.2	0.3	0.4	0.6	0.5	1.5	0.3	0.2	0.2	0.2	0.3	0.4
	3.5	3.7	0.4	0.3	0.4	0.6	0.7	1.2	2.0	2.5	3.2	0.8	0.6	1.1	0.7	0.9	0.8
180	3.5	2.3	<0.1	<0.1	<0.1	<0.1	0.2	0.3	0.2	0.5	1.1	0.9	0.7	0.6	0.3	0.5	0.2
	3.6	4.1	<0.1	0.2	0.1	0.2	0.2	0.6	1.0	1.4	3.9	2.2	2.1	1.1	1.6	1.9	1.3
215	3.5	2.5	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.1	0.3	0.6	0.6	0.2	0.1	0.1	0.2	0.1
	3.9	4.5	<0.1	<0.1	<0.1	<0.1	0.1	0.1	0.4	0.8	0.6	0.2	0.4	0.3	0.3	0.3	1.3

Table 6

Wave Heights for Plan 11 for Test Waves from 140, 125, and 70 Deg

Direction deg	Test Wave Period sec	Test Wave Height ft	Wave Height, ft, at Indicated Gage Locations 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
140	3.5	1.7	0.1	0.1	0.1	0.2	0.2	0.5	0.4	0.5	1.4	0.6	0.2	0.2	0.2	0.1	0.4
	3.5	3.7	0.2	0.3	0.3	0.6	0.6	1.2	2.8	2.7	3.7	1.4	1.2	1.1	1.3	1.3	1.4
125	3.5	2.4	<0.1	0.1	0.1	<0.1	0.2	0.2	0.3	0.7	1.9	0.5	0.5	0.7	0.9	0.9	1.4
	3.6	3.8	0.1	0.2	0.2	0.4	0.5	1.2	1.2	2.9	4.2	2.0	1.0	1.2	1.5	1.7	2.0
70	4.3	4.0	0.2	0.2	0.4	0.8	0.6	0.9	1.3	2.3	2.1	1.5	1.2	1.2	0.8	1.3	1.8
	6.0	9.0	0.9	1.1	0.8	1.4	2.3	2.7	2.7	4.5	5.5	5.2	2.5	3.6	1.5	2.1	3.0

Table 7

Wave Heights for Plans 12-18 for Test Waves from 70 Deg

Plan	Test Wave Period sec	Test Wave Height ft	Wave Height, ft, at Indicated Gage Locations 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
12	4.3	4.0	0.3	0.3	0.1	0.5	0.3	0.8	1.5	1.4	1.9	1.8	1.1	0.8	0.9	0.8	1.1
	6.0	9.0	0.5	0.8	0.9	1.3	1.4	1.7	2.1	4.3	4.8	3.0	1.6	1.2	2.3	1.3	3.2
13	4.3	4.0	0.2	0.2	0.2	0.7	0.3	0.8	0.7	2.7	1.8	1.3	0.8	1.1	1.0	0.6	1.4
	6.0	9.0	0.6	1.0	0.7	1.3	1.5	1.7	2.0	3.4	4.6	2.9	1.4	1.8	2.0	1.0	2.3
14	4.3	4.0	0.2	0.2	0.1	0.4	0.1	0.8	0.9	1.0	1.8	0.9	1.0	0.5	0.4	0.7	0.5
	6.0	9.0	0.6	0.9	0.5	1.2	1.3	1.2	1.9	2.9	4.2	2.9	2.2	2.2	1.5	1.0	1.1
15	4.3	4.0	0.2	0.2	0.1	0.2	0.1	0.3	0.2	0.6	1.8	0.2	0.4	0.2	0.3	0.1	0.2
	6.0	9.0	0.2	0.2	0.1	0.3	0.5	0.4	0.8	1.3	5.5	1.3	0.6	0.8	0.7	0.7	1.2
16	4.3	4.0	0.3	0.4	0.3	0.5	0.5	0.8	0.6	1.5	2.8	1.2	1.5	0.7	1.0	0.9	0.9
	6.0	9.0	0.3	0.3	0.5	0.5	0.6	0.7	0.6	3.5	5.0	2.0	1.0	0.9	1.5	1.1	2.6
17	4.3	4.0	0.2	0.3	0.3	0.8	0.5	0.8	1.0	3.0	3.0	1.5	0.8	0.7	1.4	0.9	1.2
	6.0	9.0	0.9	1.3	1.0	2.1	1.9	1.9	2.6	3.3	4.7	3.3	2.4	2.2	2.9	1.2	2.9
18	4.3	4.0	0.2	0.2	0.4	0.4	0.3	0.8	0.5	2.4	2.3	2.0	1.0	0.7	1.0	0.9	1.3
	6.0	9.0	0.5	0.5	0.8	0.9	0.7	0.9	1.4	2.4	6.0	2.7	1.3	0.7	1.8	1.5	2.2

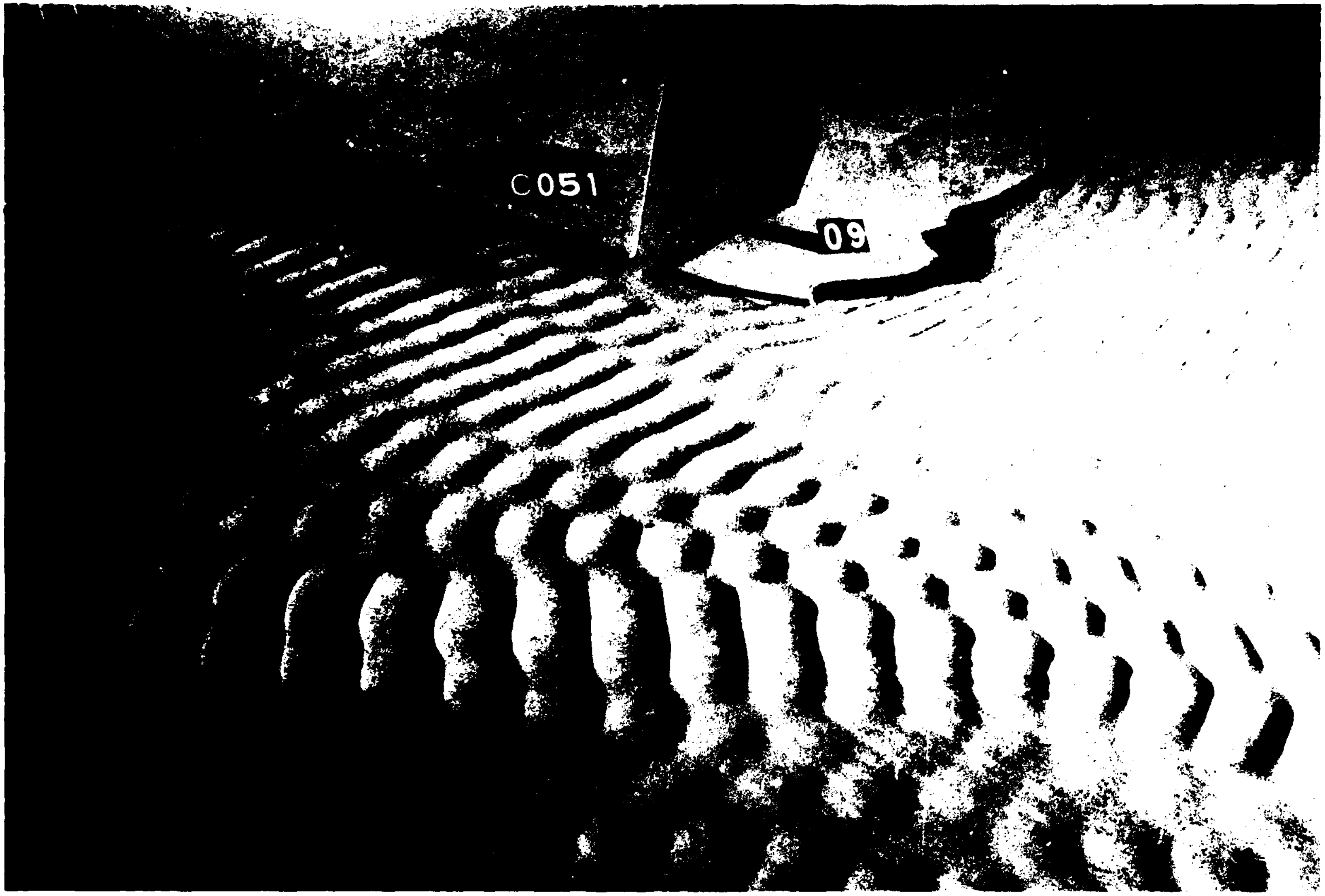

Photo 1. Typical wave patterns for existing conditions; 4.3-sec, 4.0-ft waves from 70 deg

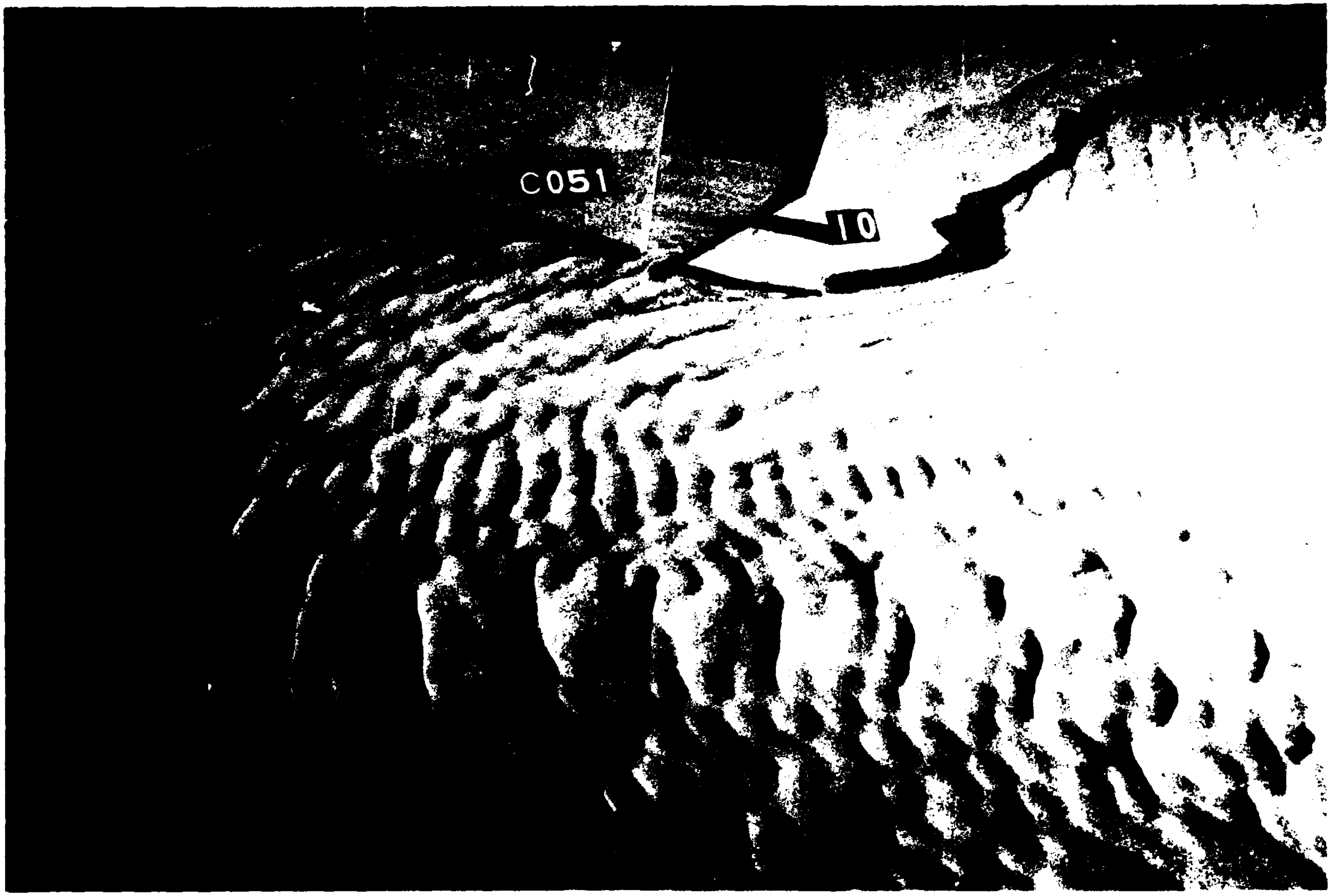

Photo 2. Typical wave patterns for existing conditions; 6.0-sec, 9.0-ft waves from 70 deg

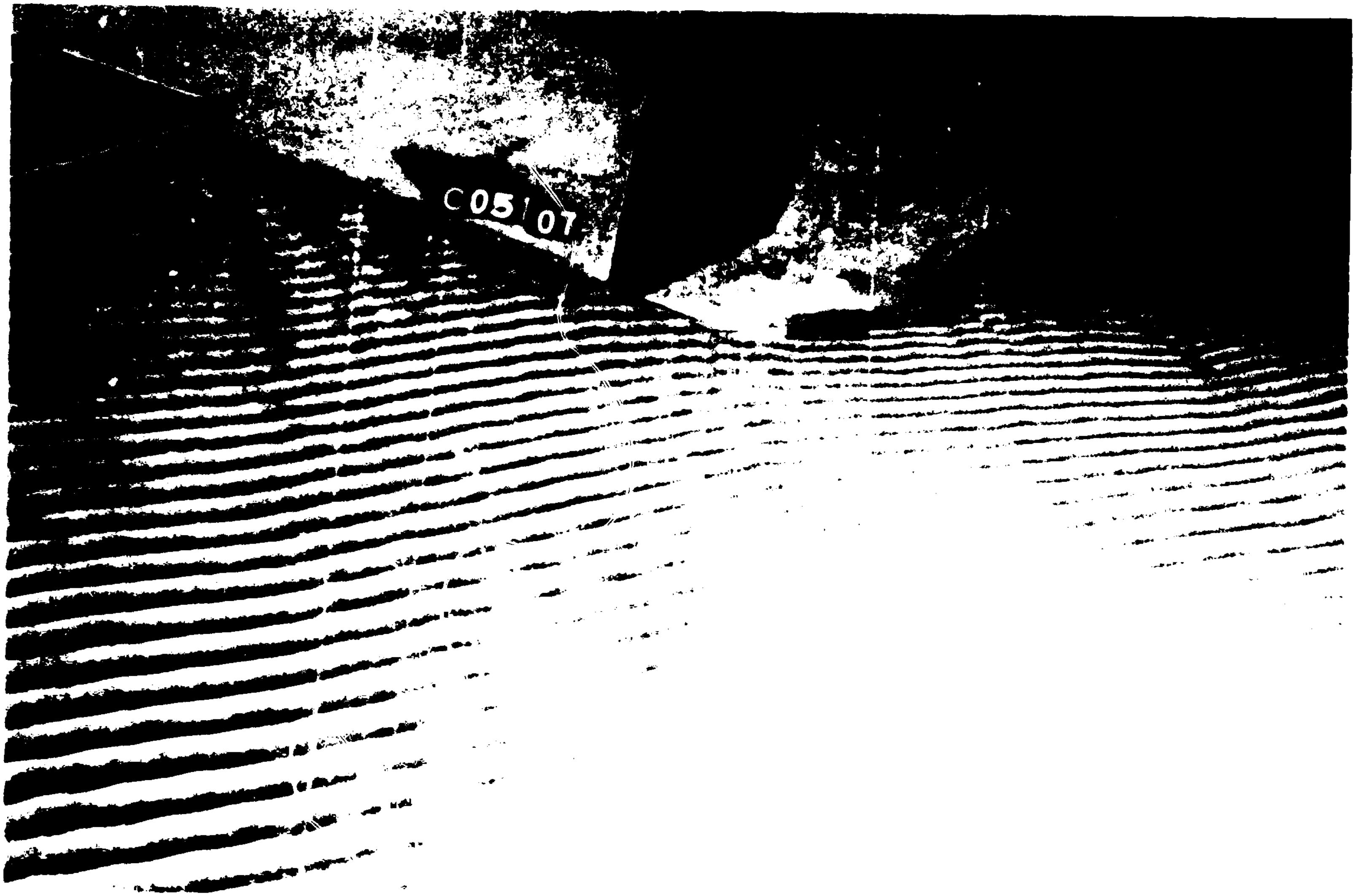

Photo 3. Typical wave patterns for existing conditions; 3.5-sec, 2.4-ft waves from 125 deg

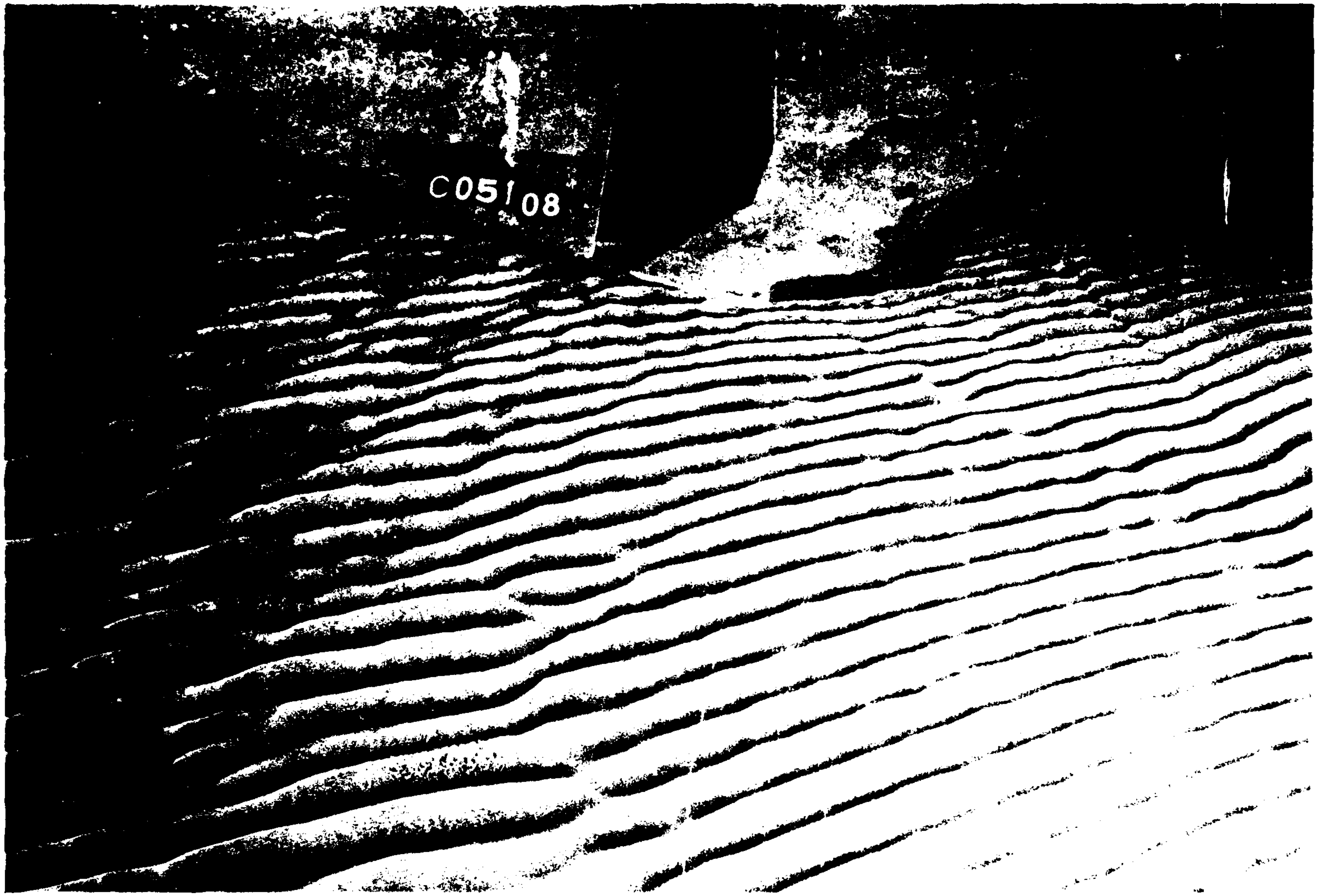

Photo 4. Typical wave patterns for existing conditions; 3.6-sec, 3.8-ft waves from 125 deg

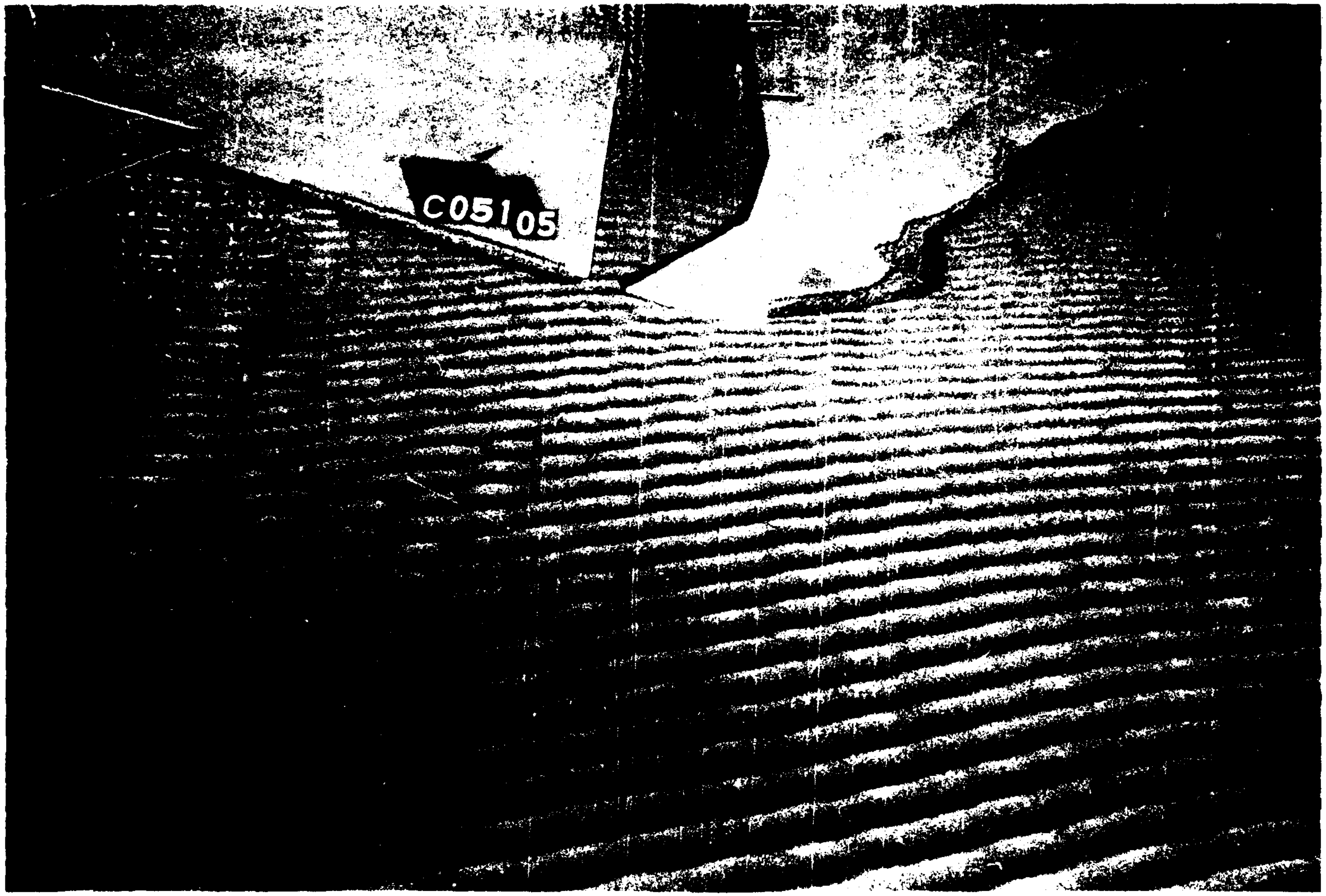

Photo 5. Typical wave patterns for existing conditions; 3.5-sec, 1.7-ft waves from 140 deg

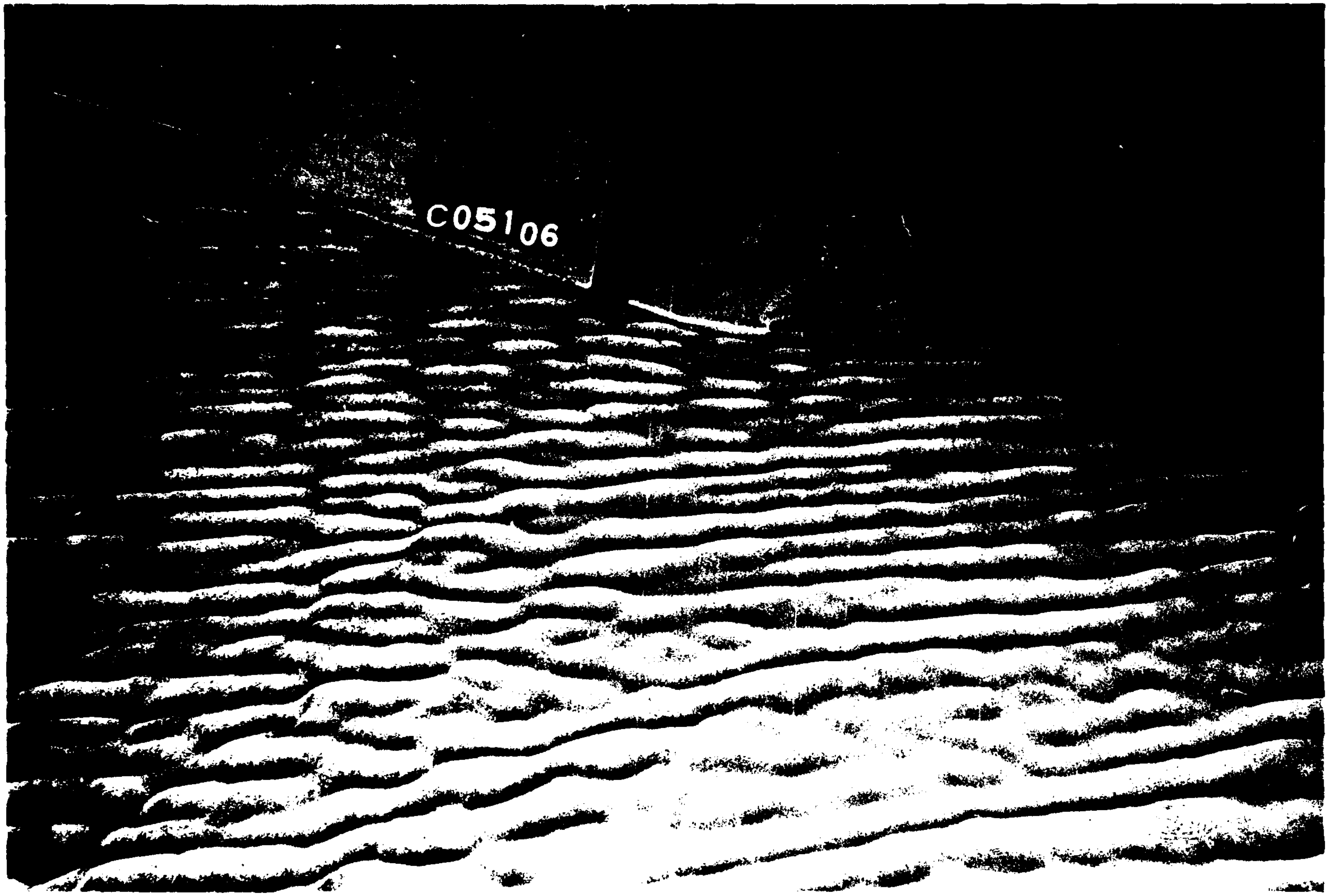

Photo 6. Typical wave patterns for existing conditions; 3.5-sec, 3.7-ft waves from 140 deg

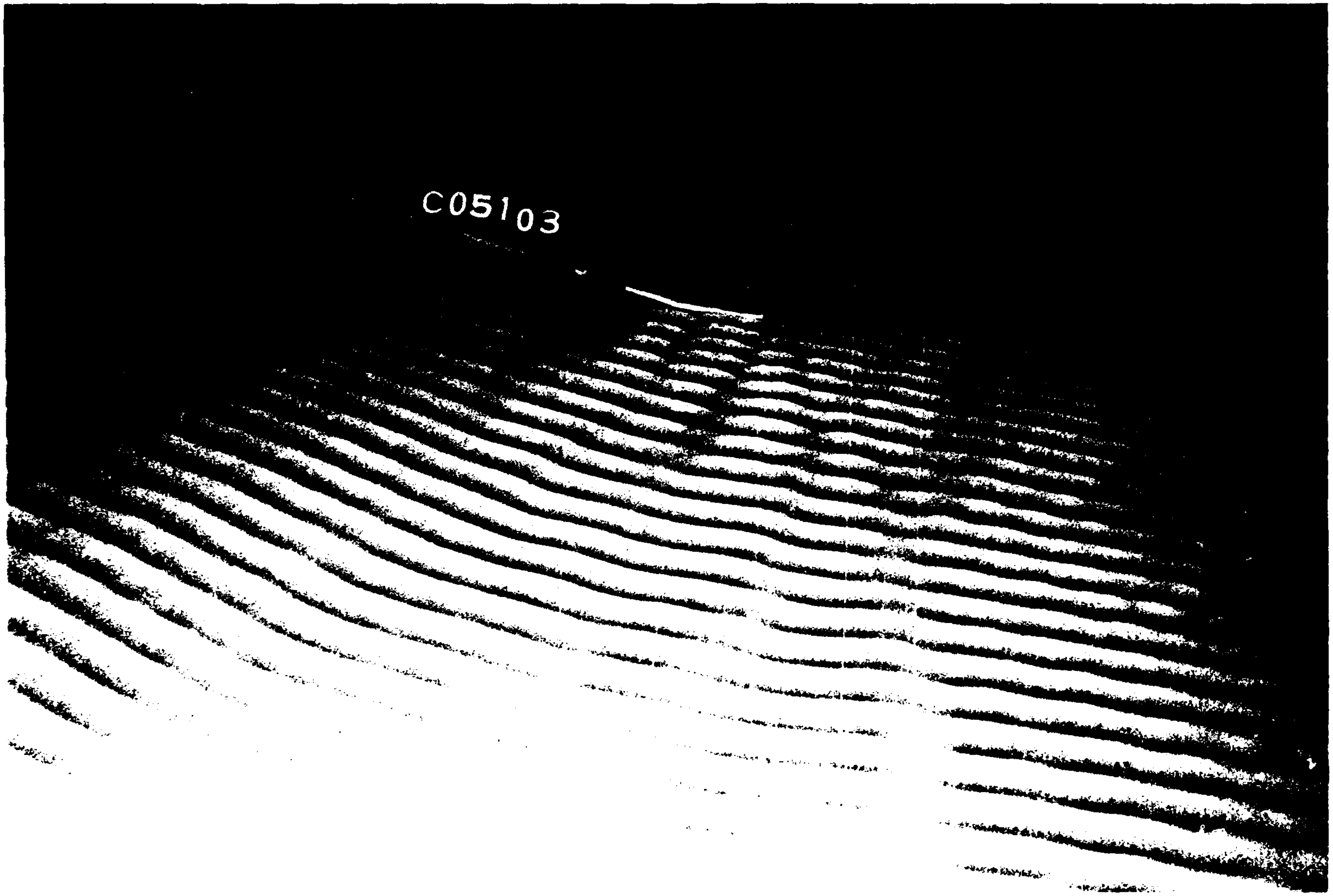

Photo 7. Typical wave patterns for existing conditions; 3.5-sec, 2.3-ft waves from 180 deg

Photo 8. Typical wave patterns for existing conditions; 3.6-sec, 4.1-ft waves from 180 deg

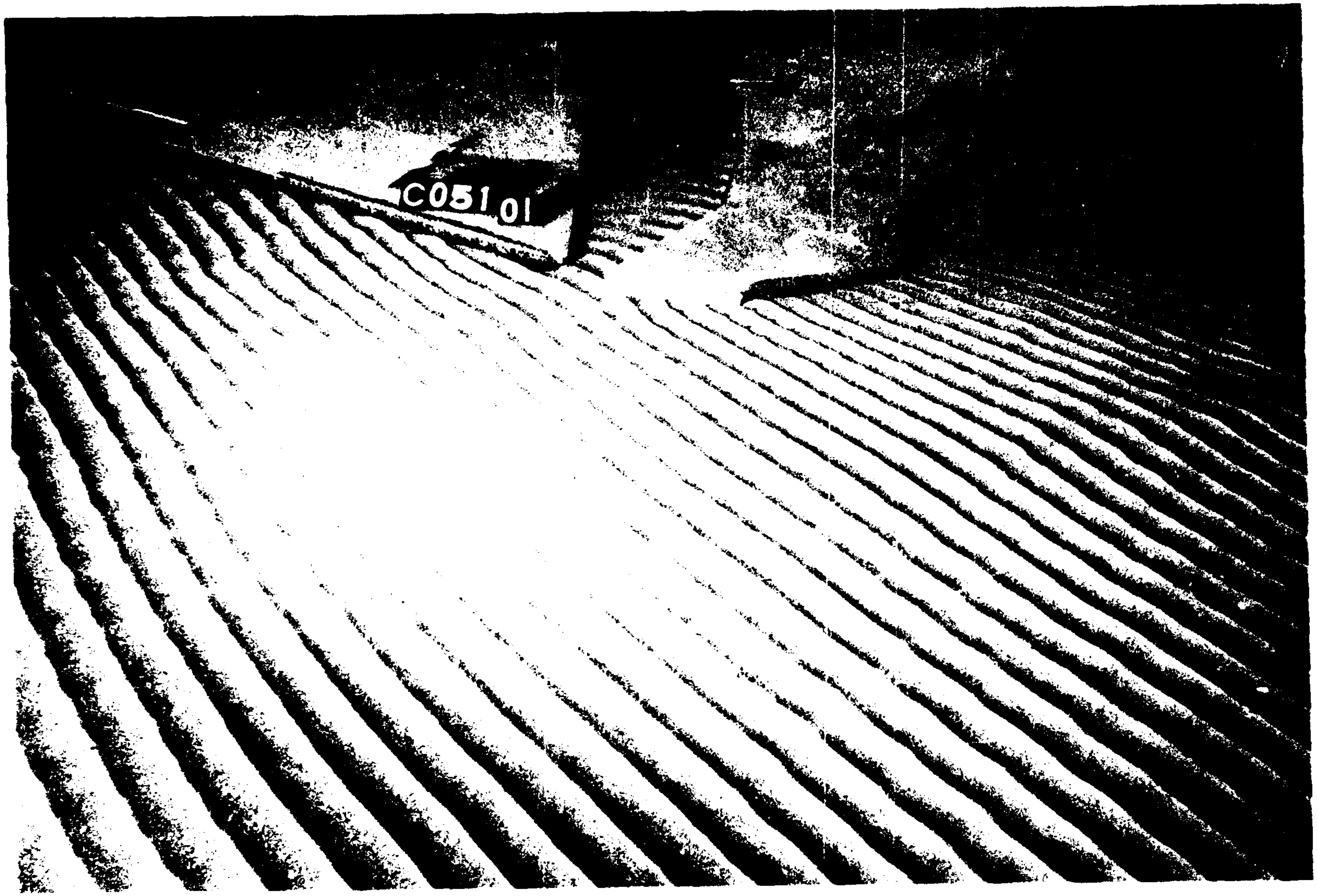

Photo 9. Typical wave patterns for existing conditions; 3.5-sec, 2.5-ft waves from 215 deg

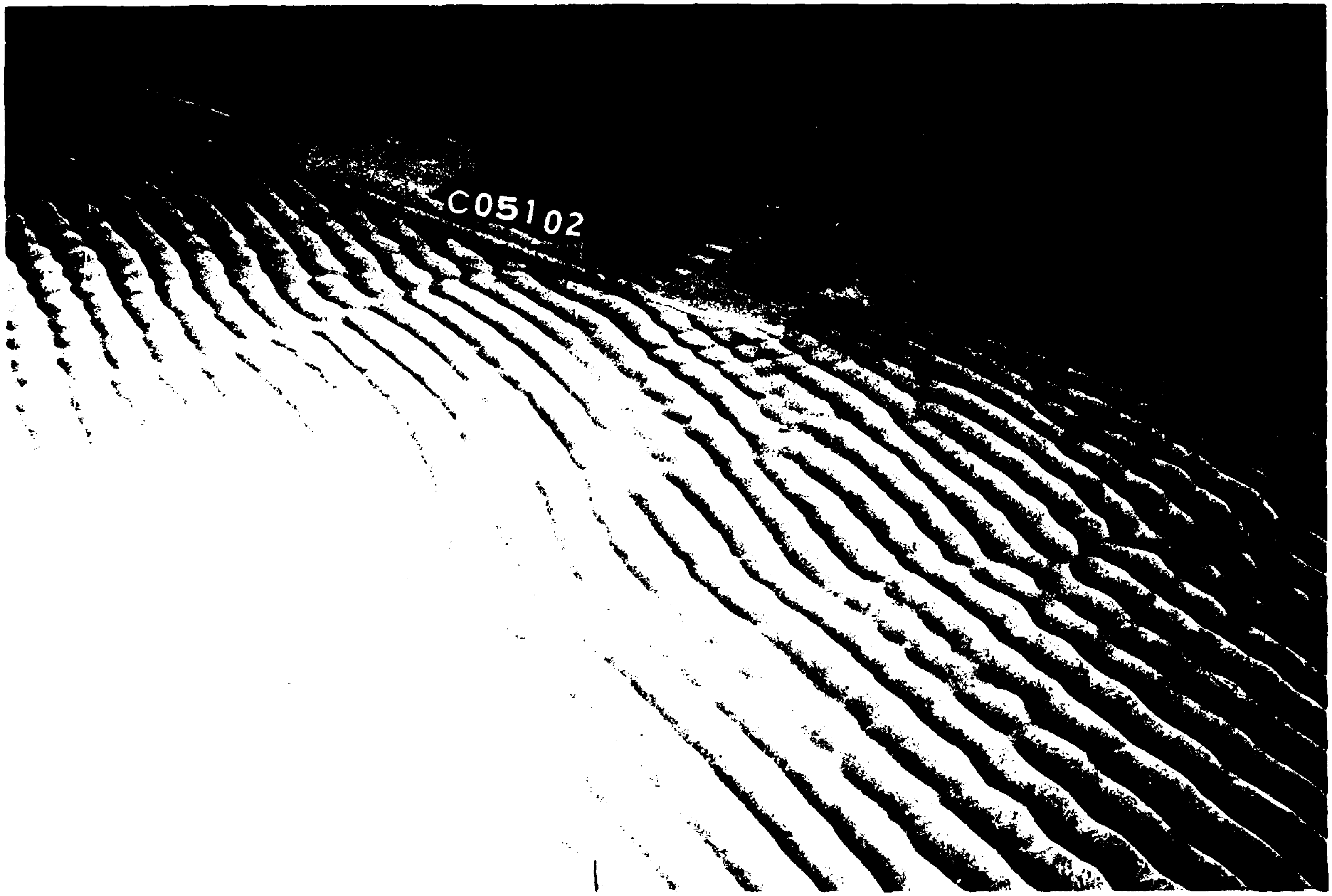

Photo 10. Typical wave patterns for existing conditions; 3.9-sec, 4.5-ft waves from 215 deg

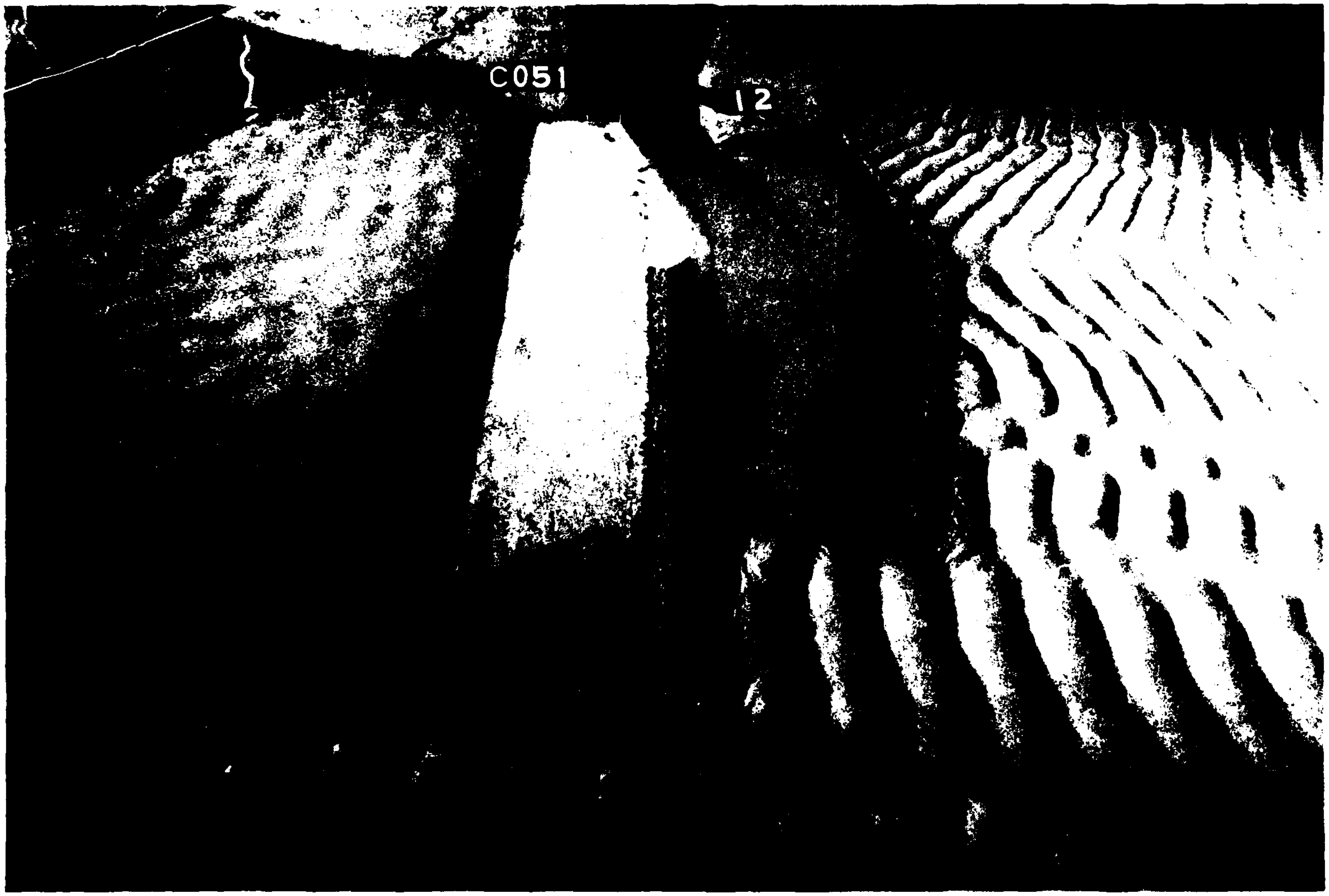

Photo 11. Typical wave patterns for Plan 1; 4.3-sec, 4.0-ft waves from 70 deg

Photo 12. Typical wave patterns for Plan 1; 6.0-sec, 9.0-ft waves from 70 deg

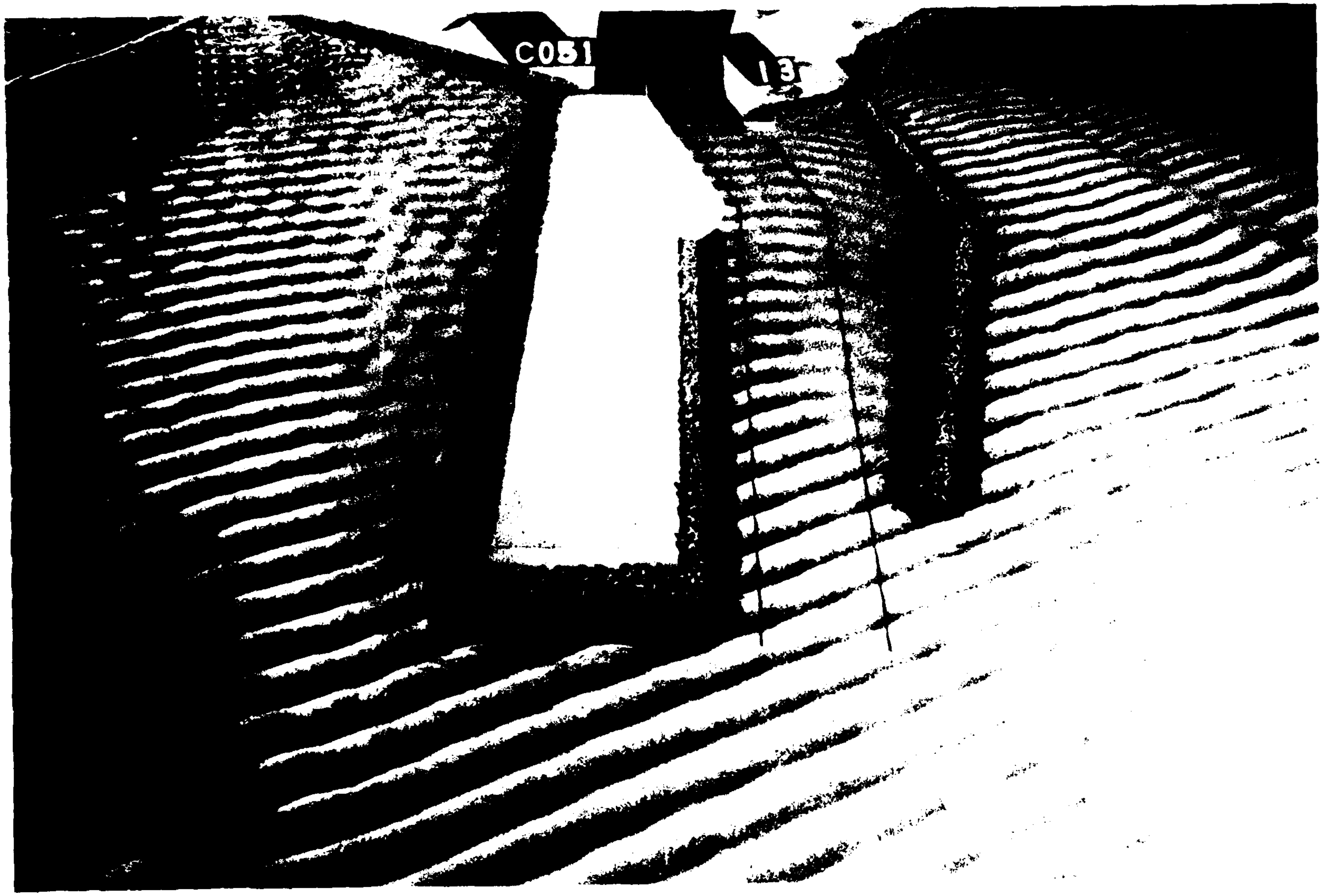

Photo 13. Typical wave patterns for Plan 1; 3.5-sec, 2.4-ft waves from 125 deg

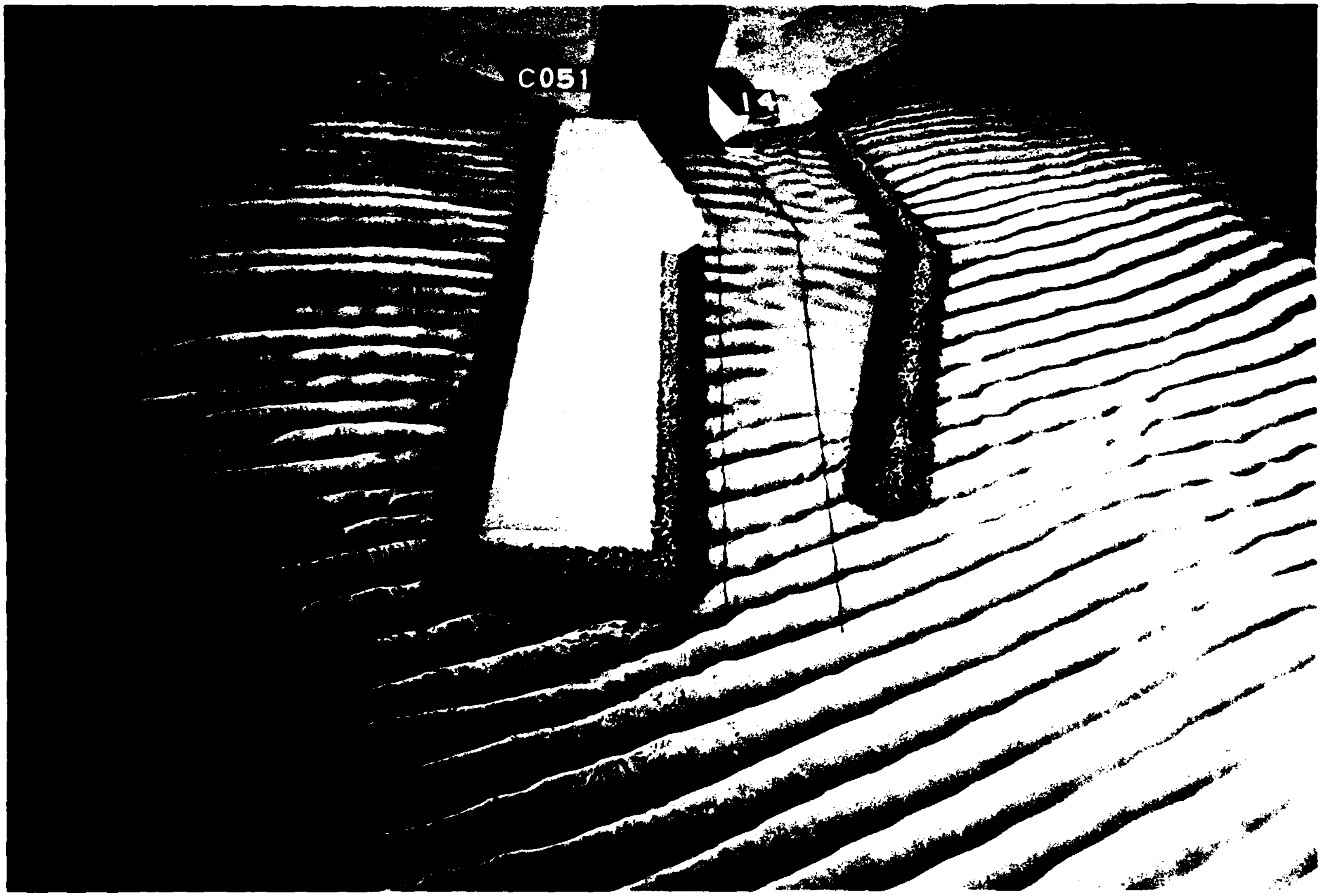

Photo 14. Typical wave patterns for Plan 1; 3.6-sec, 3.8-ft waves from 125 deg

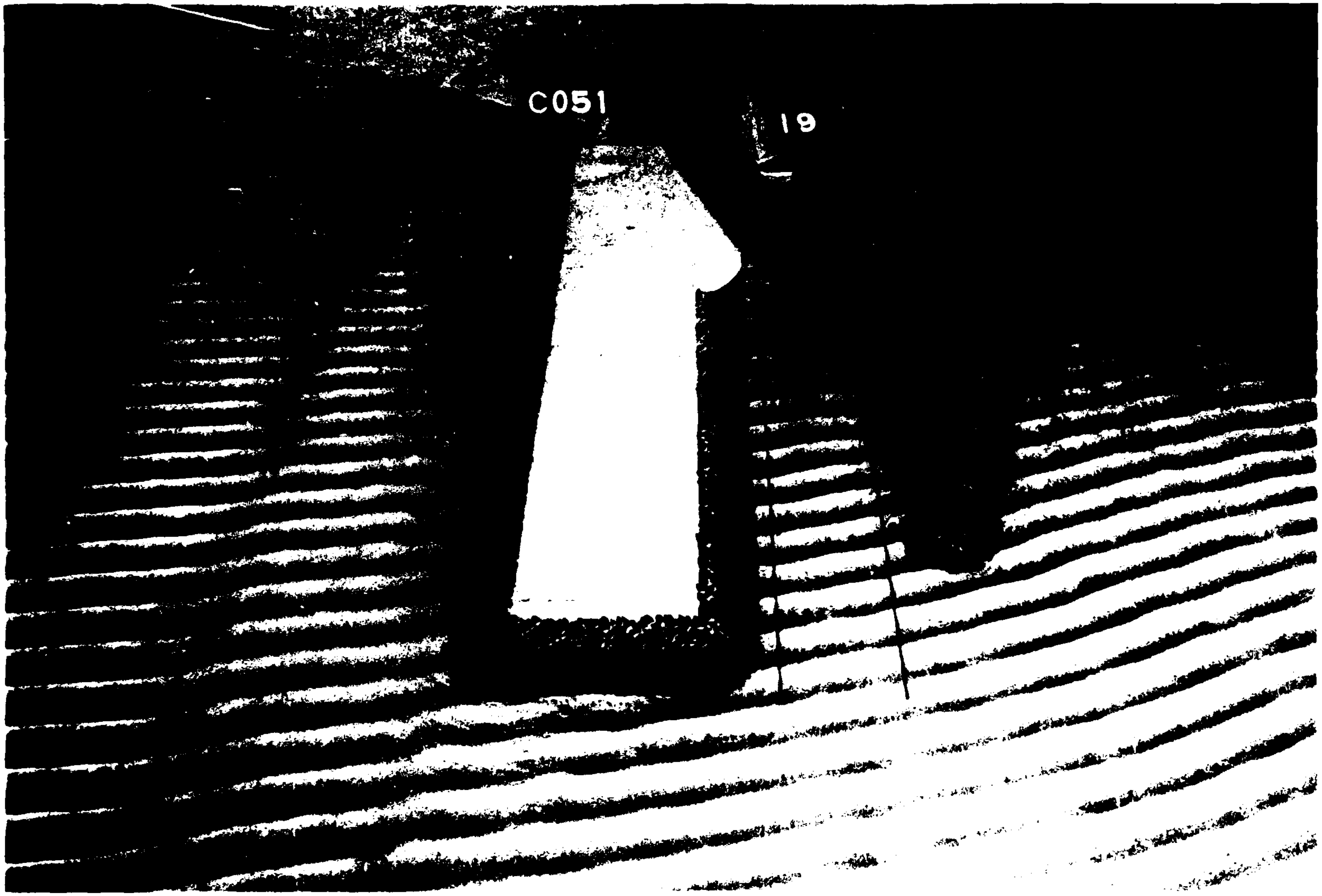

Photo 15. Typical wave patterns for Plan 1; 3.5-sec, 1.7-ft waves from 140 deg

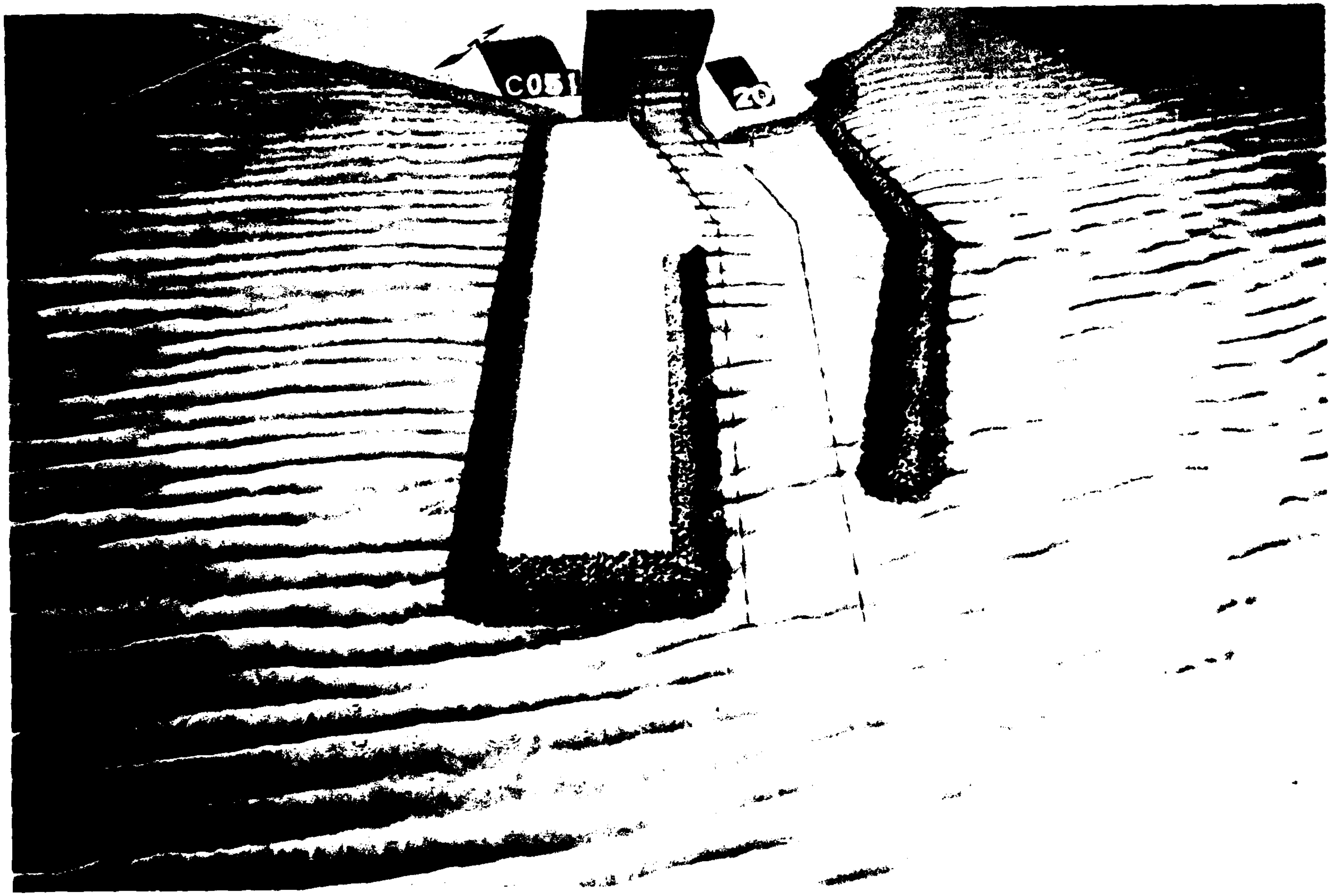

Photo 16. Typical wave patterns for Plan 1; 3.5-sec, 3.7-ft waves from 140 deg

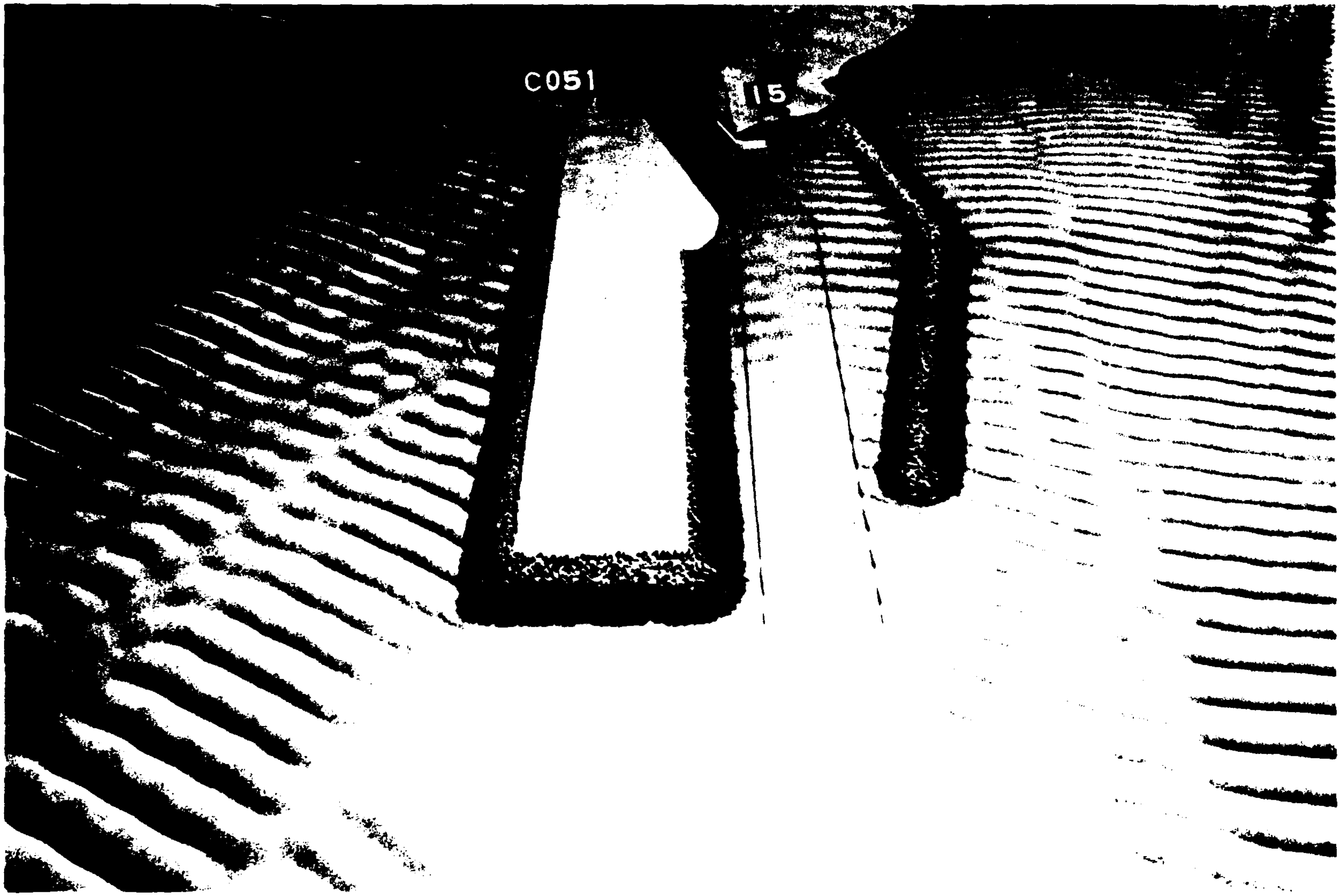

Photo 17. Typical wave patterns for Plan 1; 3.5-sec, 2.3-ft waves from 180 deg

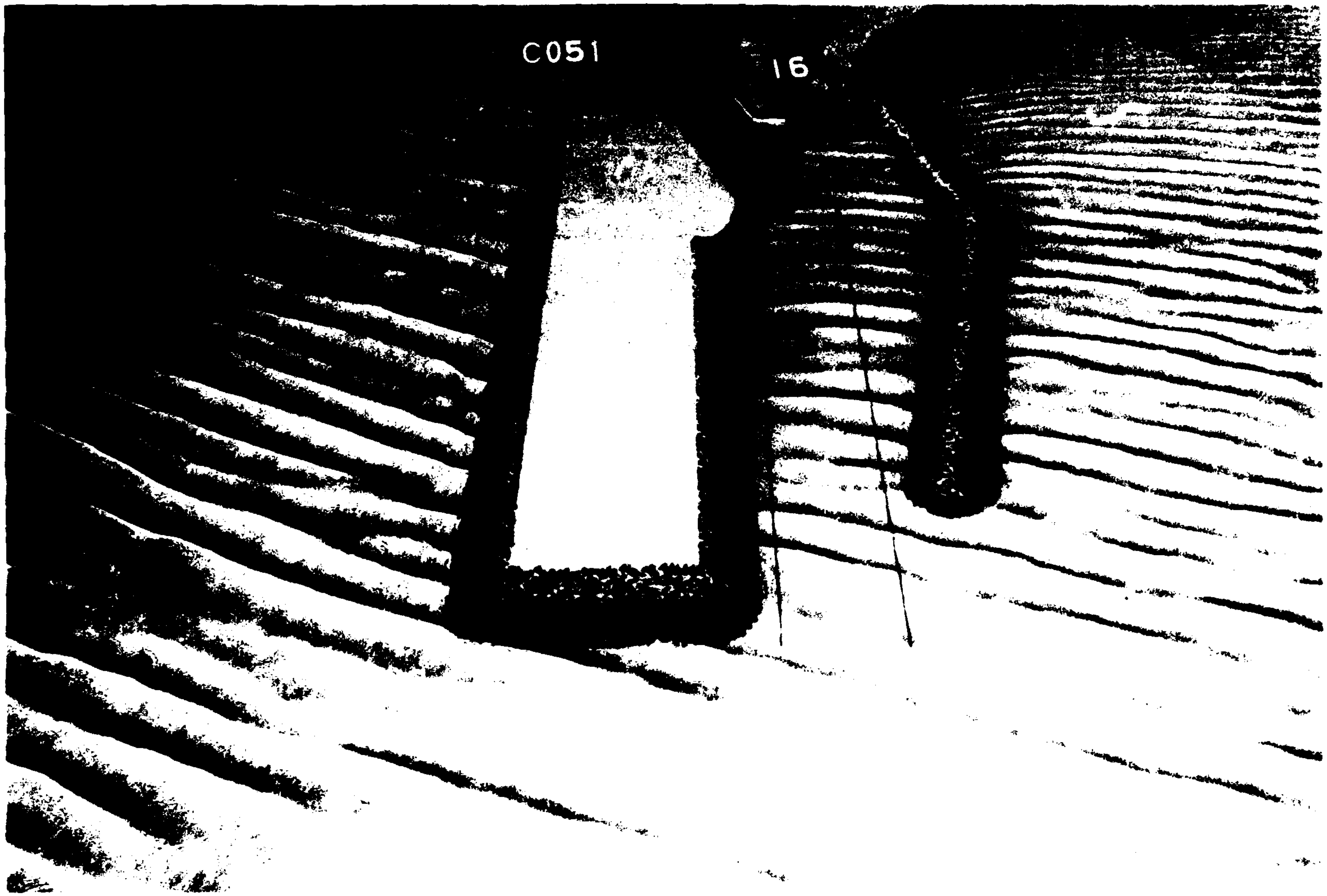

Photo 18. Typical wave patterns for Plan 1; 3.6-sec, 4.1-ft waves from 180 deg

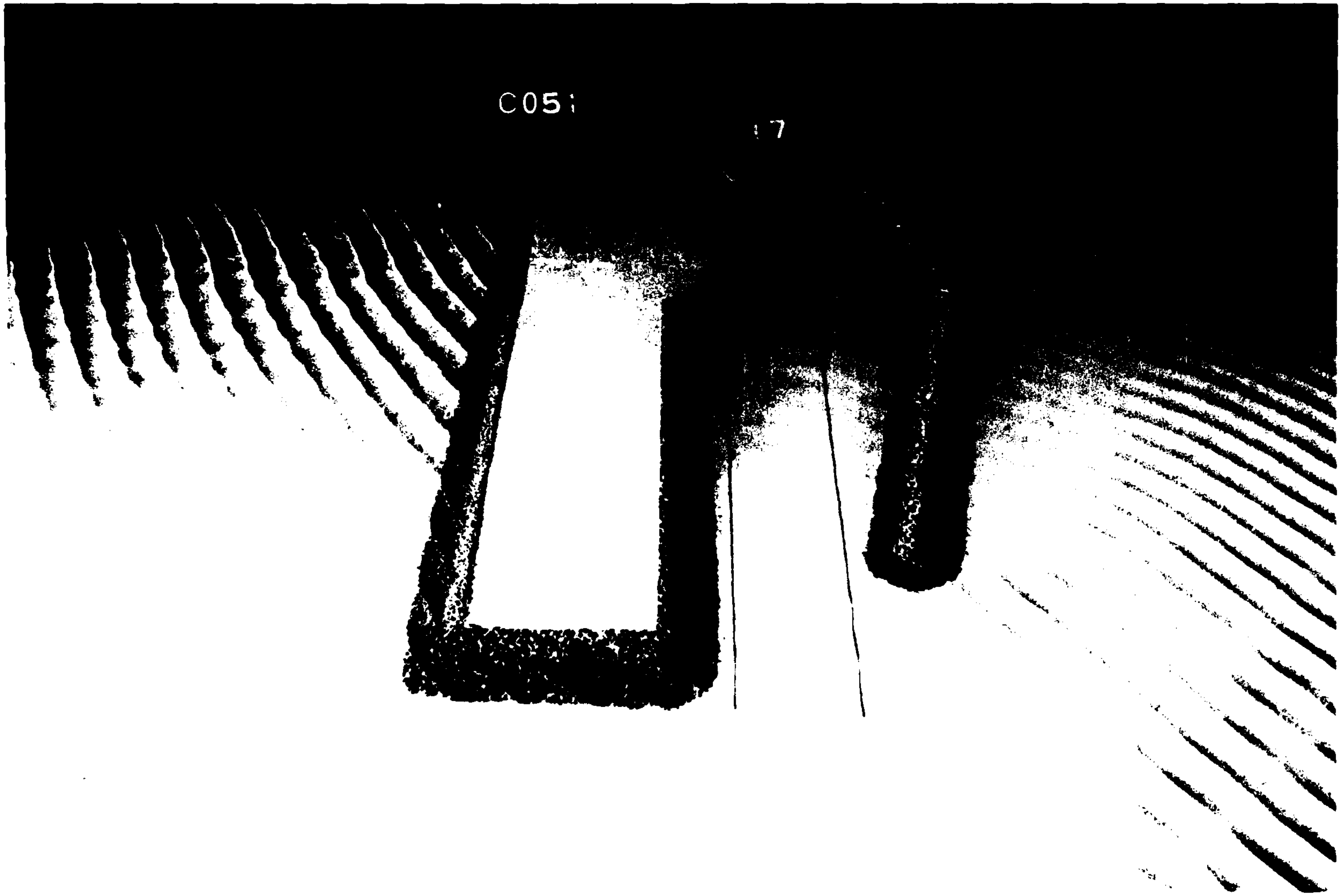

Photo 19. Typical wave patterns for Plan 1; 3.5-sec, 2.5-ft waves from 215 deg

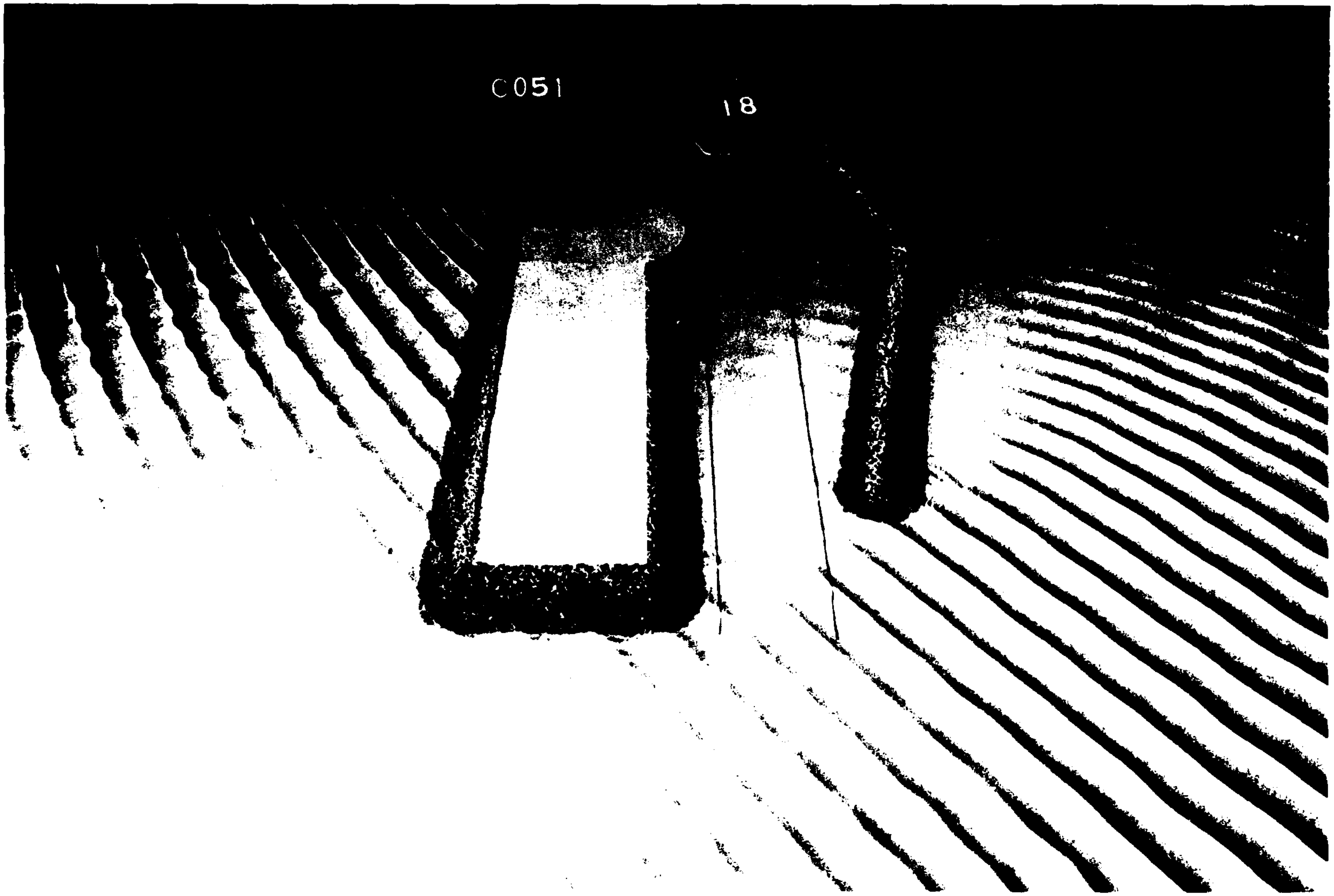

Photo 20. Typical wave patterns for Plan 1; 3.9-sec, 4.5-ft waves from 215 deg

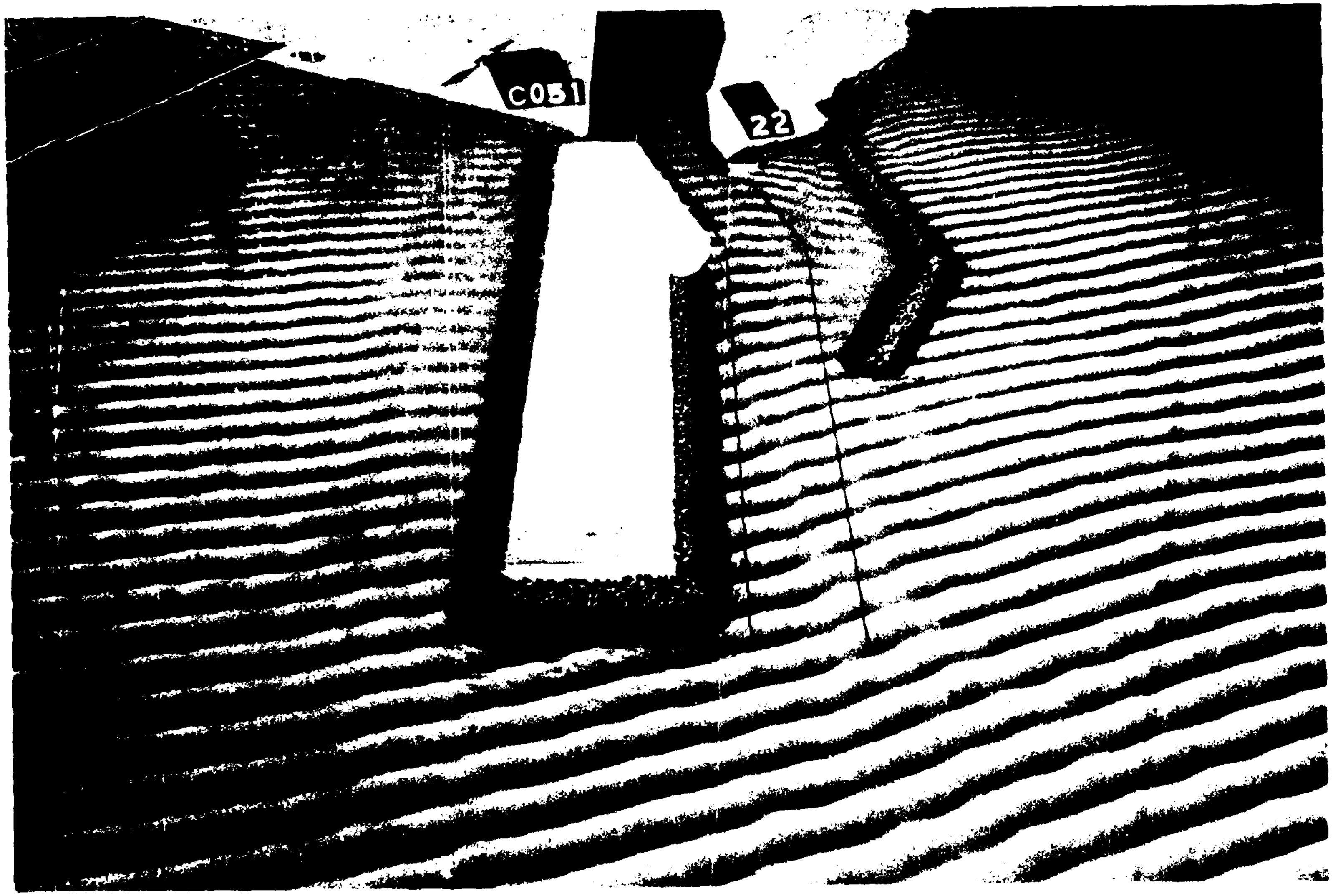

Photo 21. Typical wave patterns for Plan 2; 3.5-sec, 1.7-ft waves from 140 deg

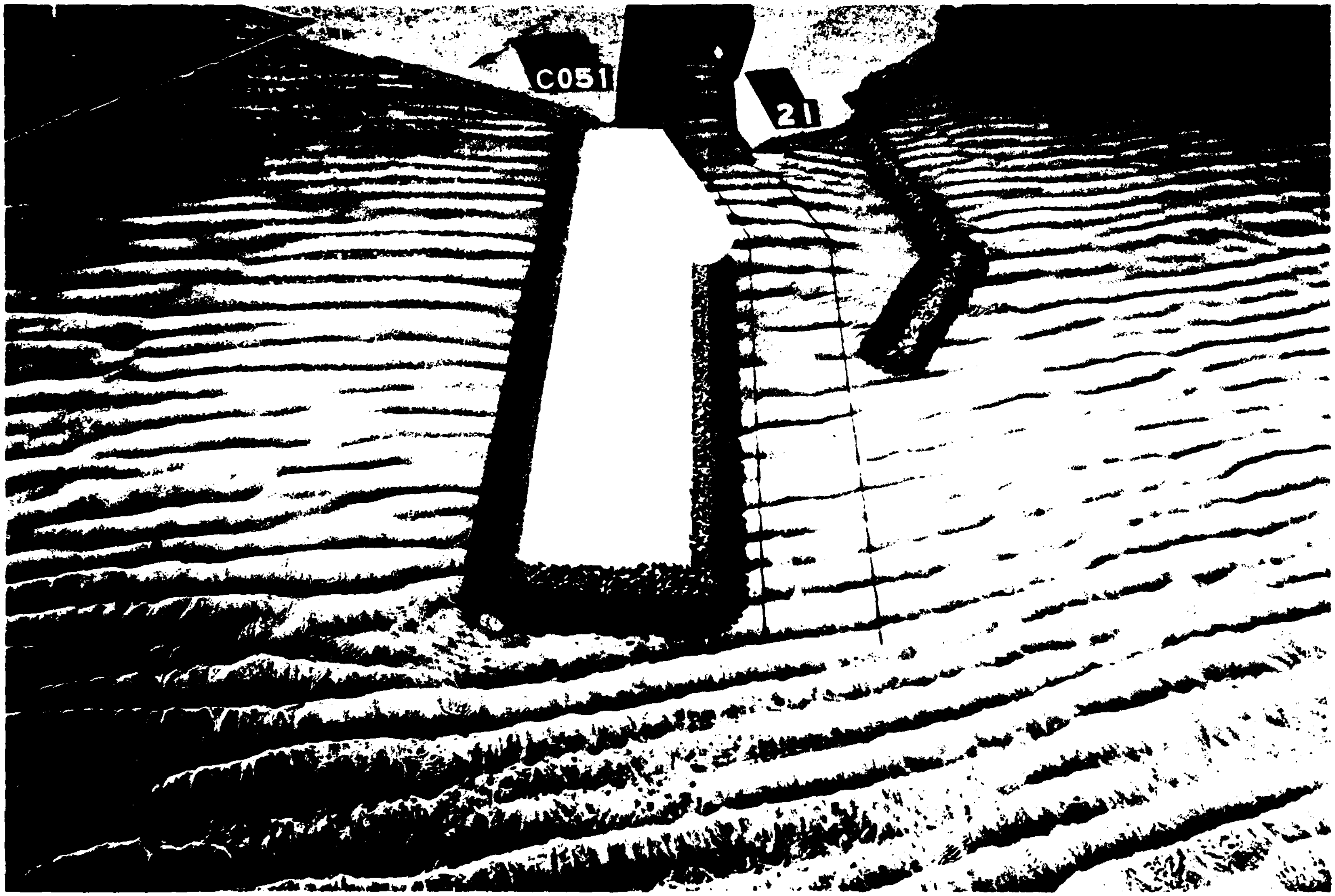

Photo 22. Typical wave patterns for Plan 2; 3.5-sec, 3.7-ft waves from 140 deg

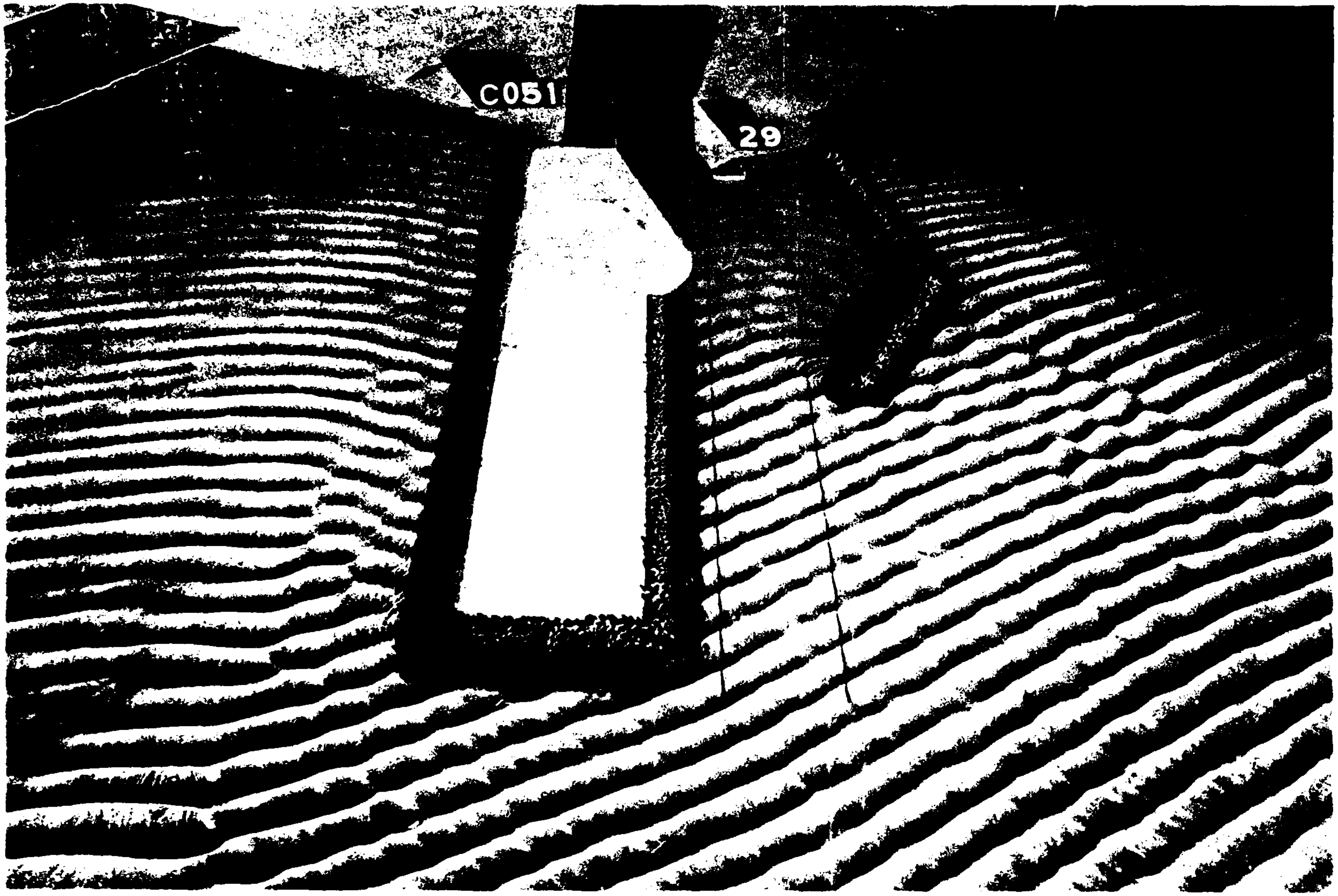

Photo 23. Typical wave patterns for Plan 2; 3.5-sec, 2.4-ft waves from 125 deg

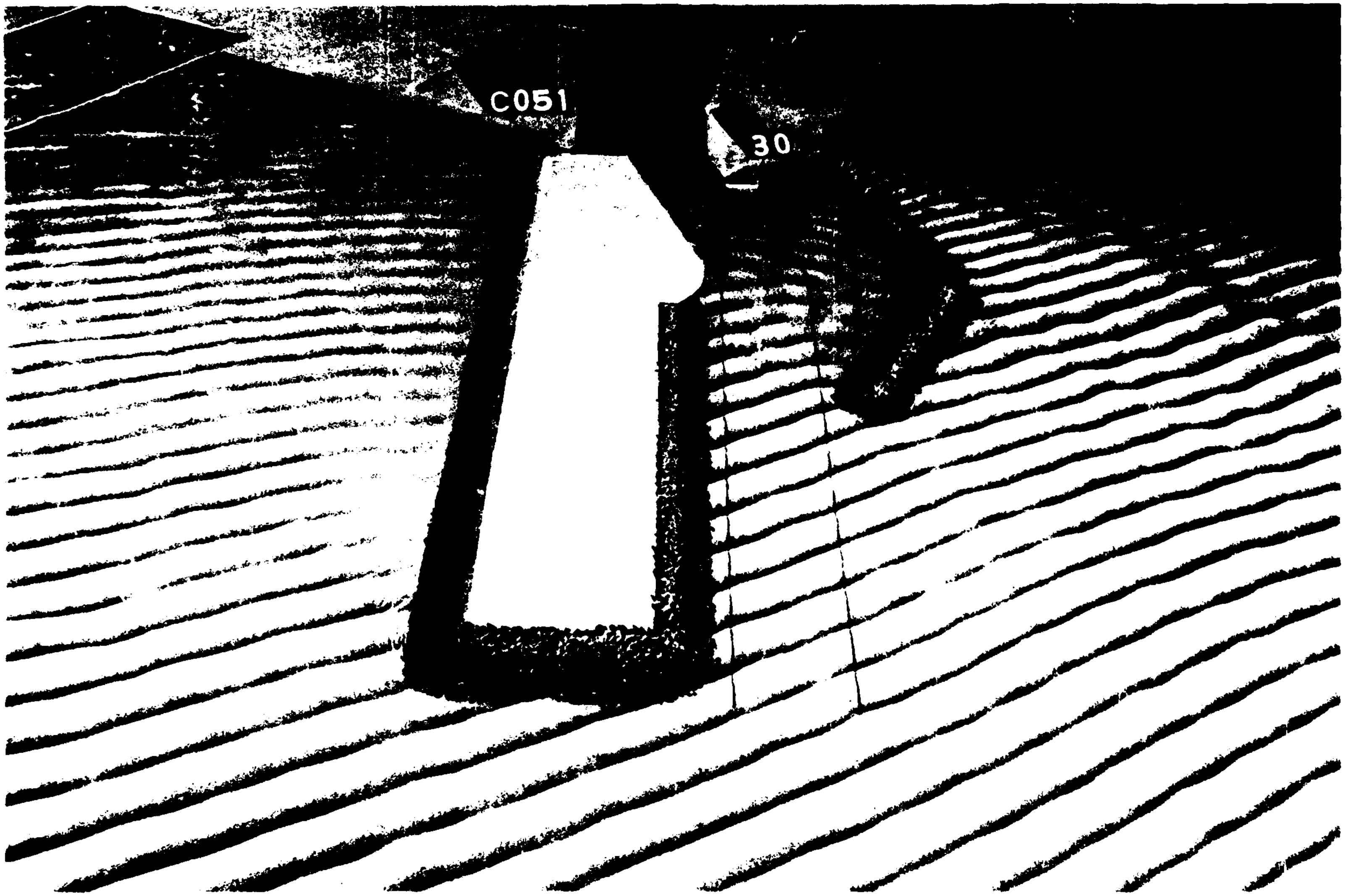

Photo 24. Typical wave patterns for Plan 2; 3.6-sec, 3.8-ft waves from 125 deg

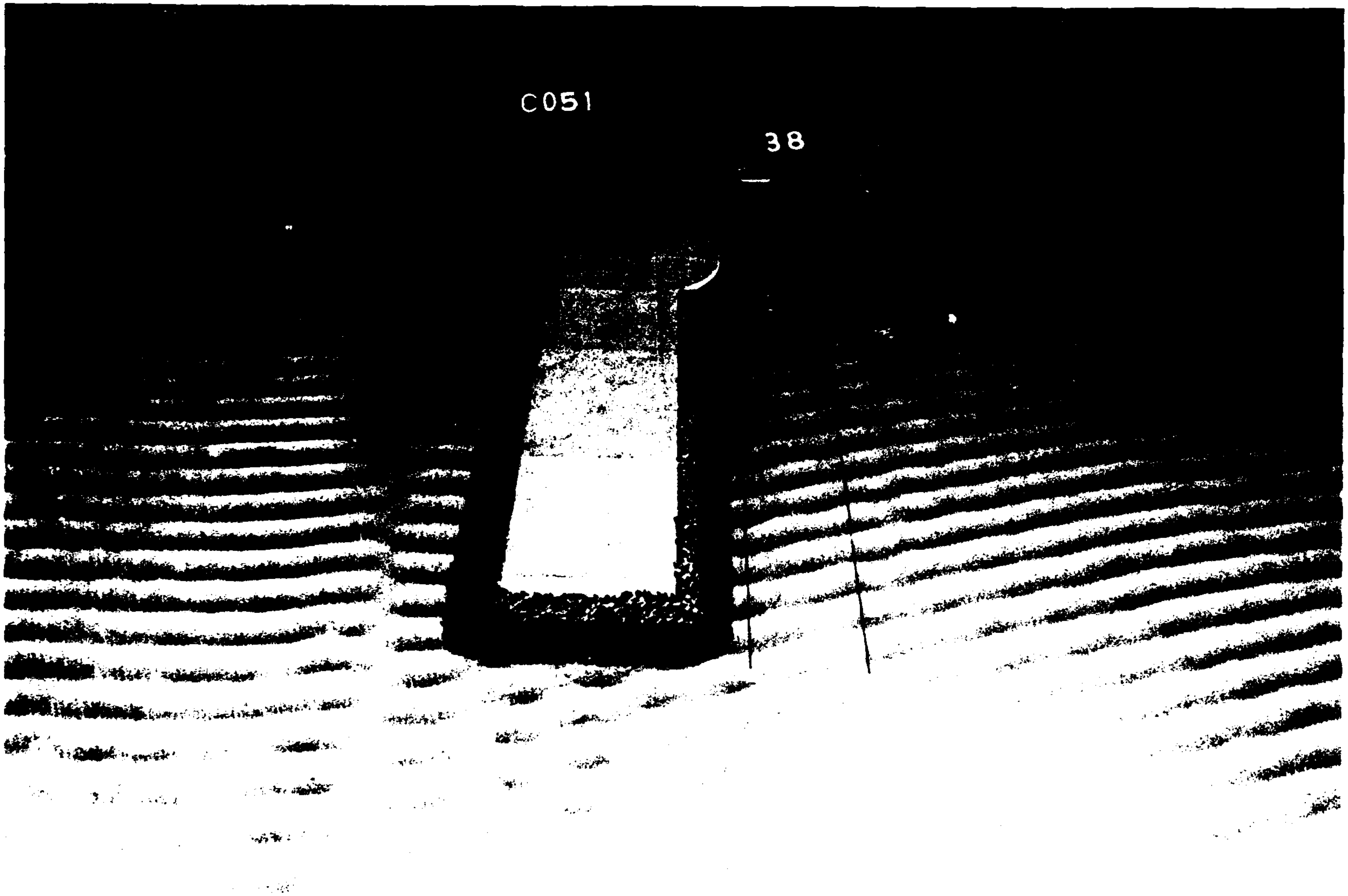

Photo 25. Typical wave patterns for Plan 3; 3.5-sec, 1.7-ft waves from 140 deg

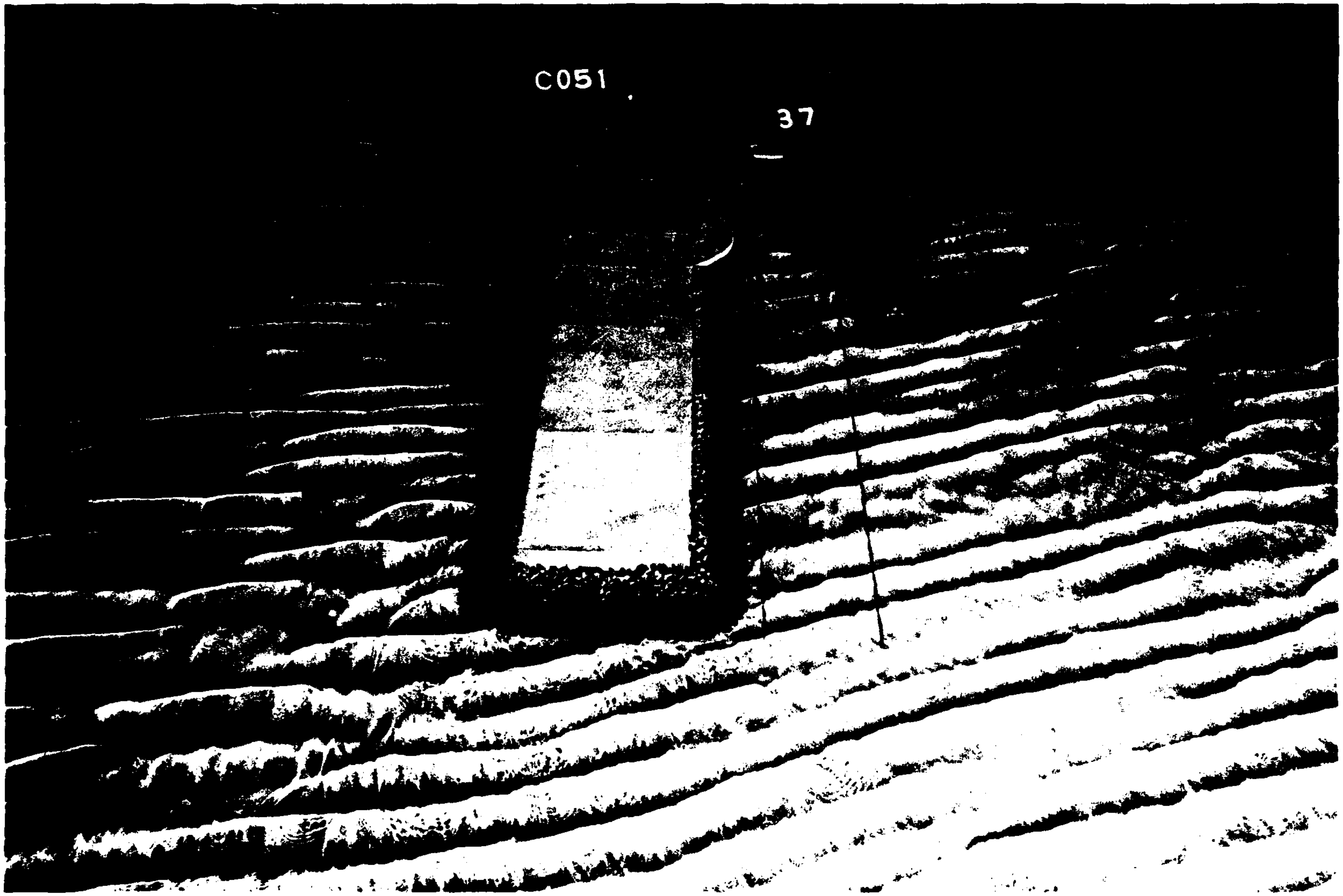

Photo 26. Typical wave patterns for Plan 3; 3.5-sec, 3.7-ft waves from 140 deg

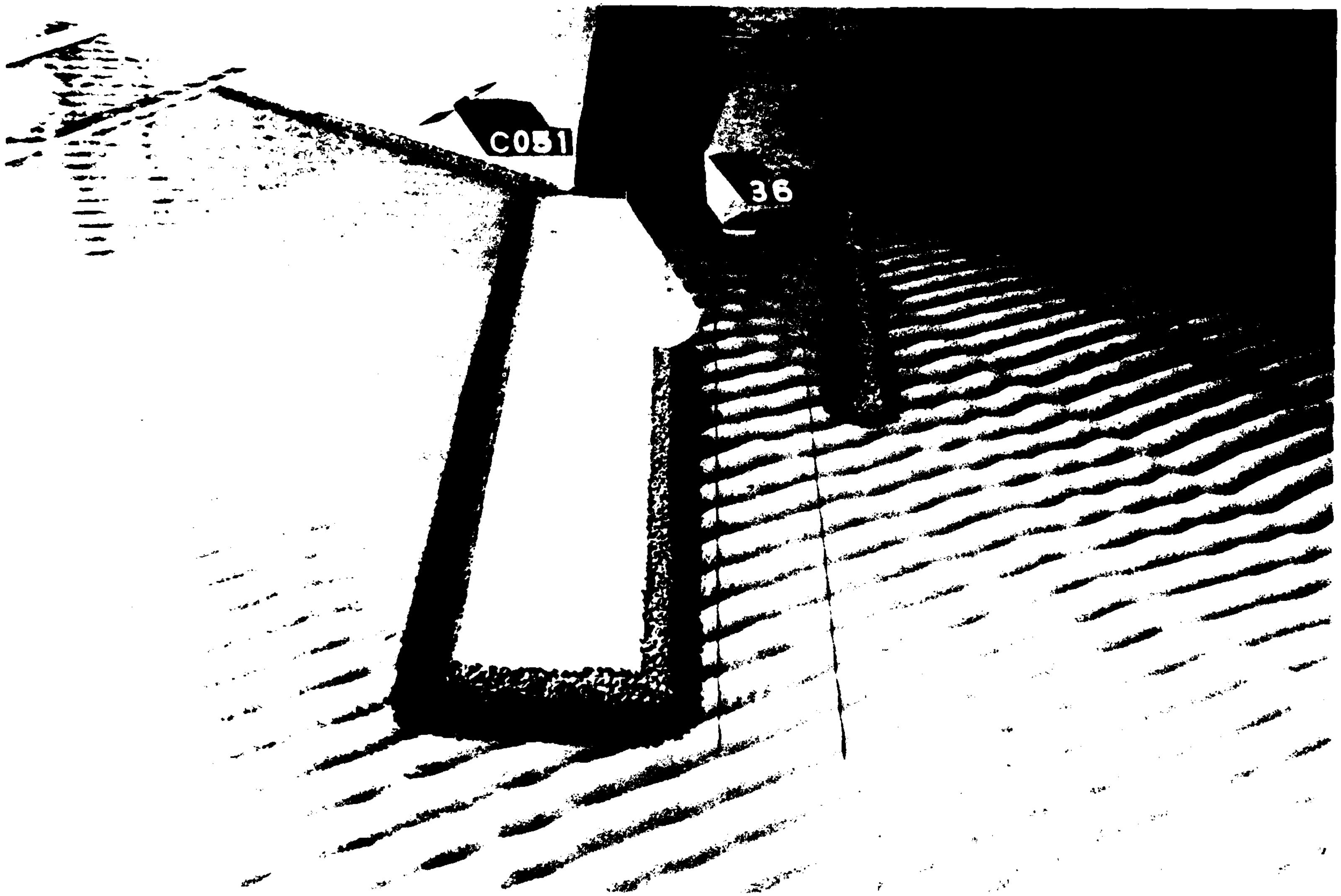

Photo 27. Typical wave patterns for Plan 3; 3.5-sec, 2.4-ft waves from 125 deg

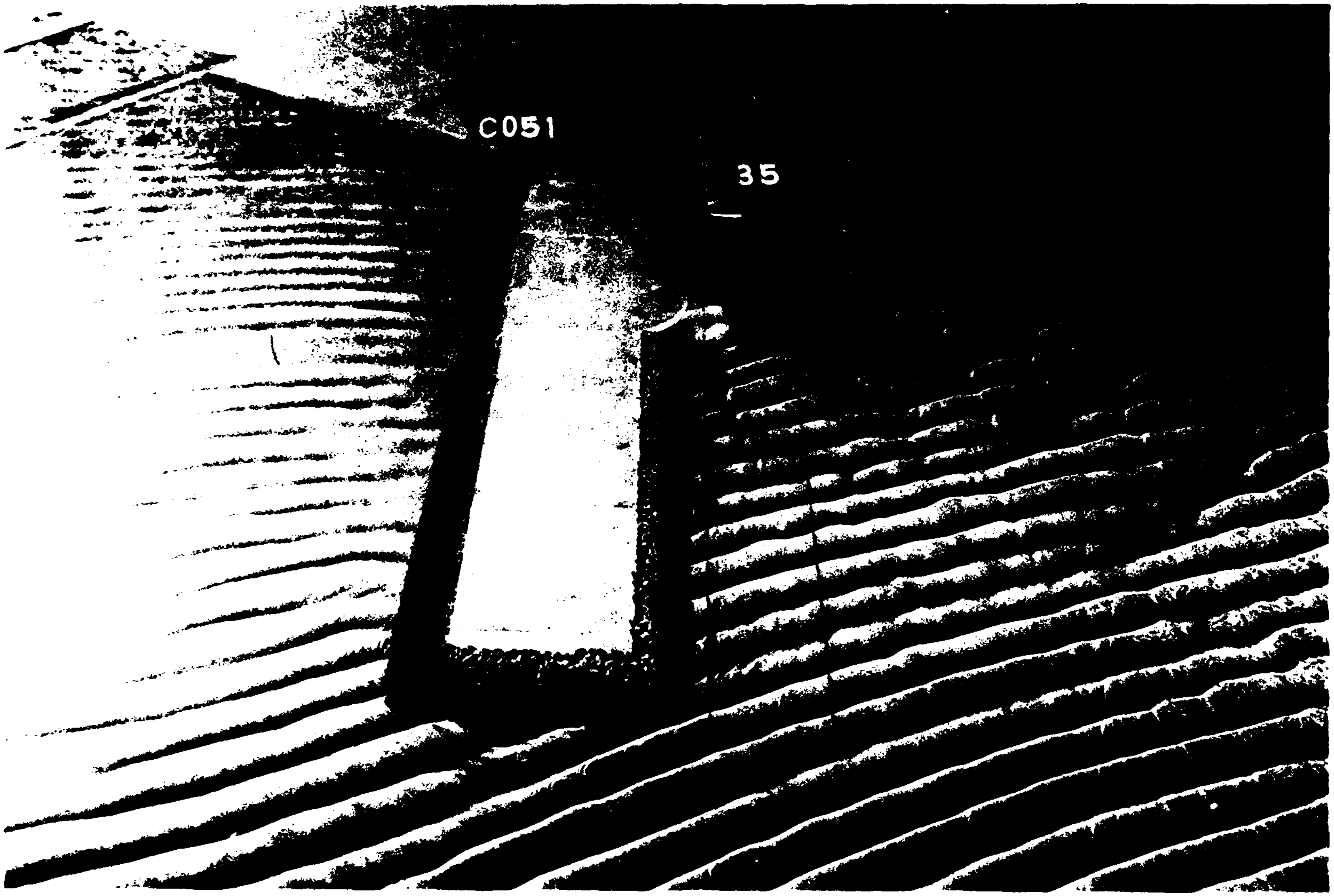

Photo 28. Typical wave patterns for Plan 3; 3.6-sec, 3.8-ft waves from 125 deg

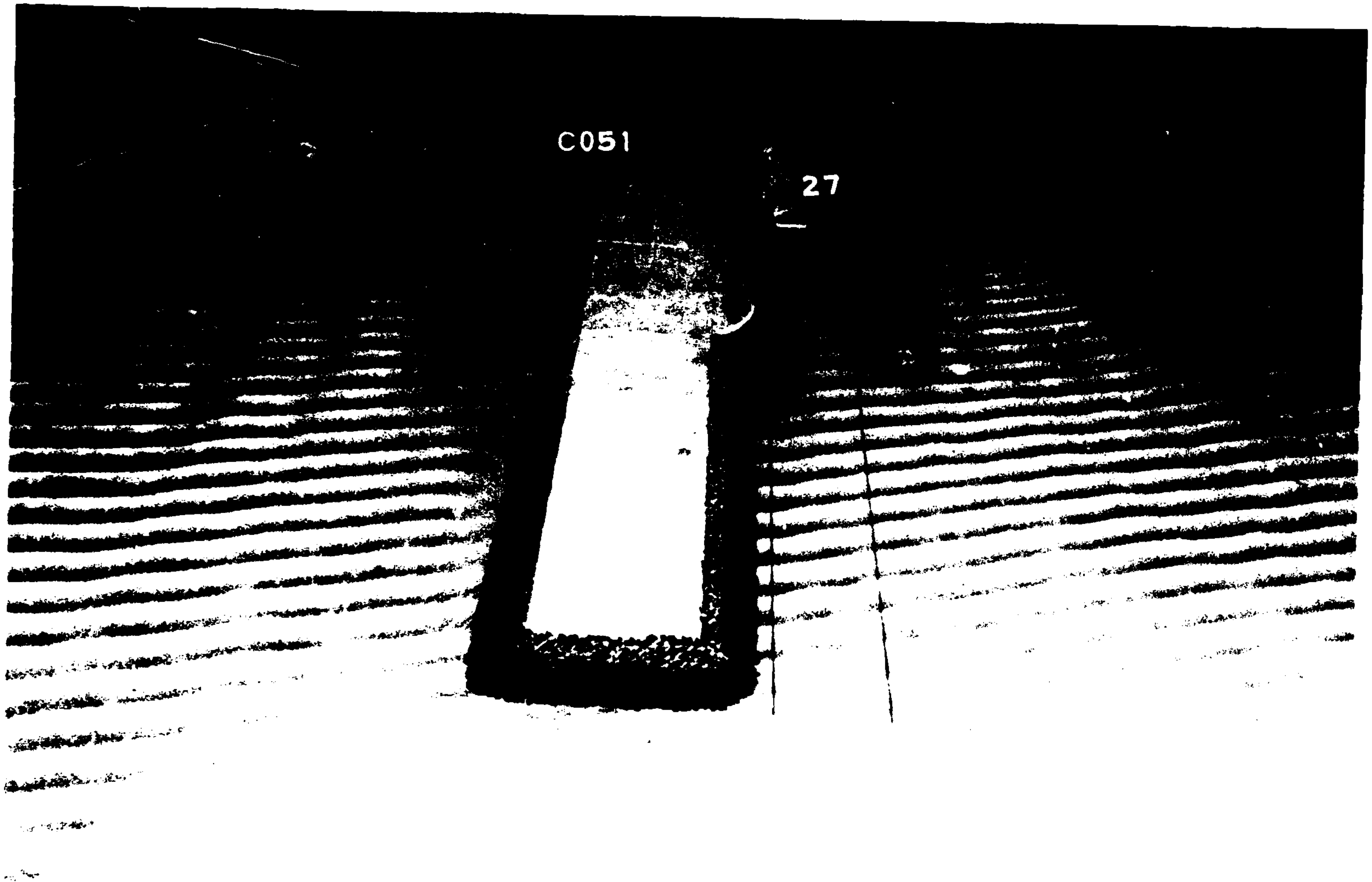

Photo 29. Typical wave patterns for Plan 4; 3.5-sec, 1.7-ft waves from 140 deg

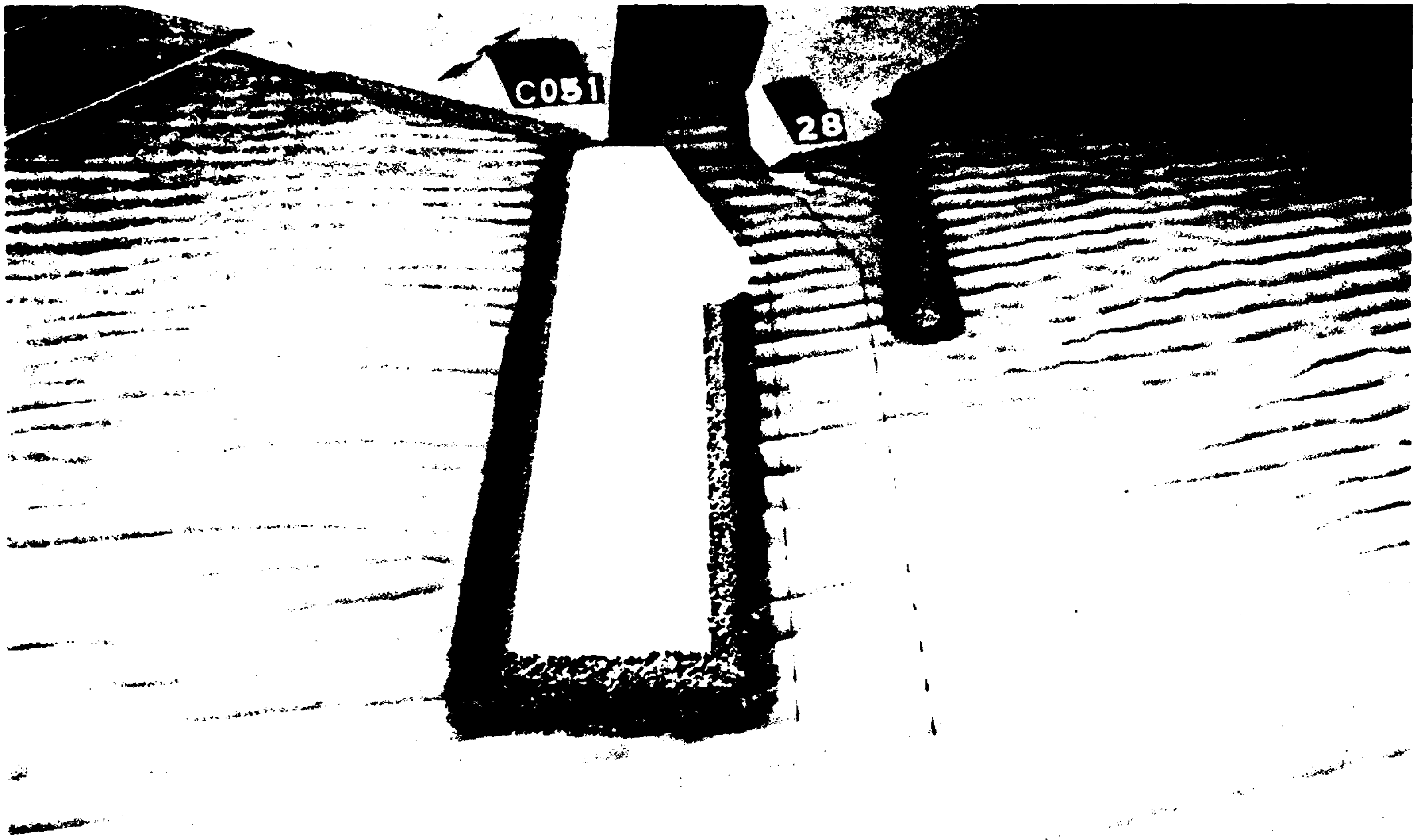

Photo 30. Typical wave patterns for Plan 4; 3.5-sec, 3.7-ft waves from 140 deg

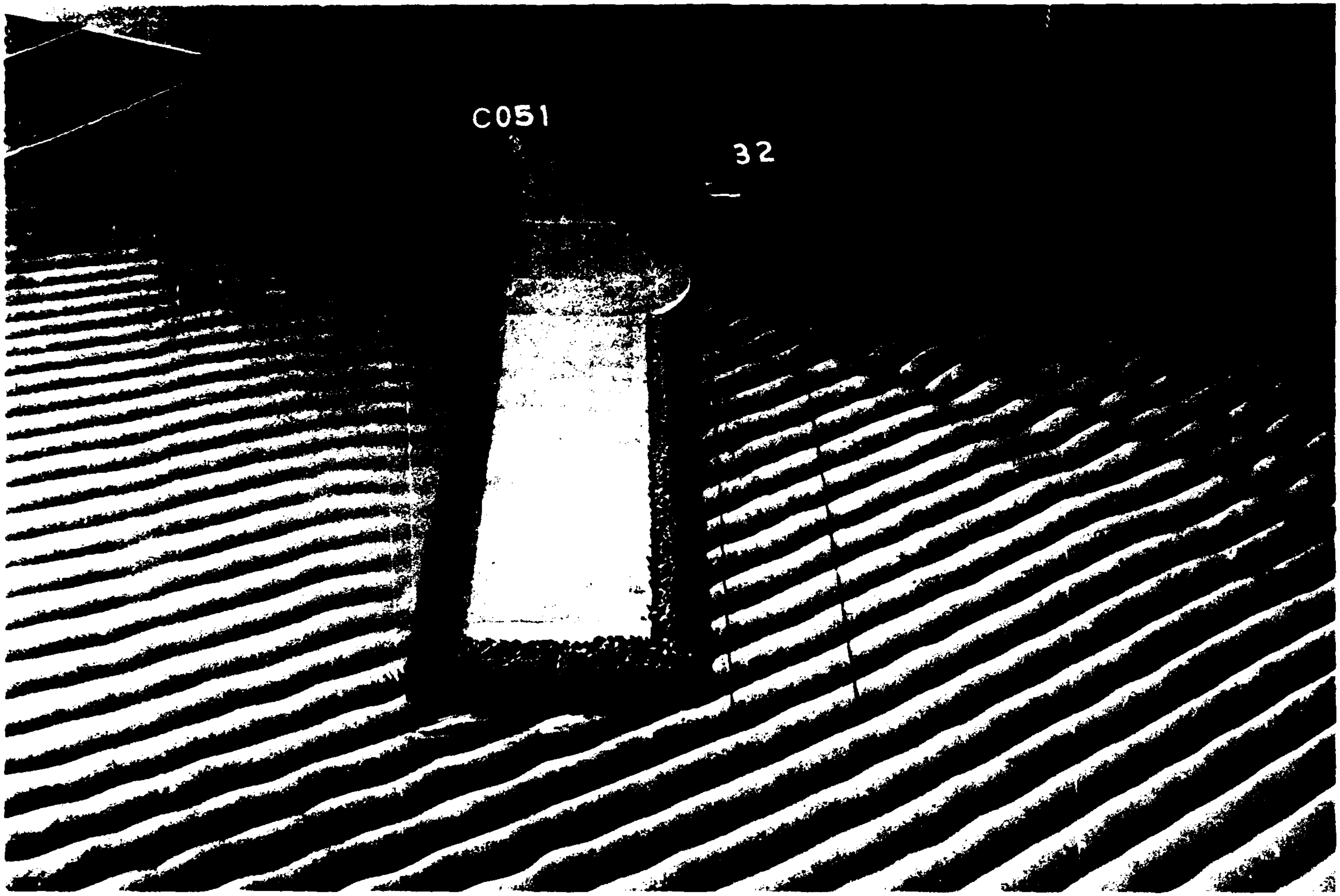

Photo 31. Typical wave patterns for Plan 4; 3.5-sec, 2.4-ft waves from 125 deg

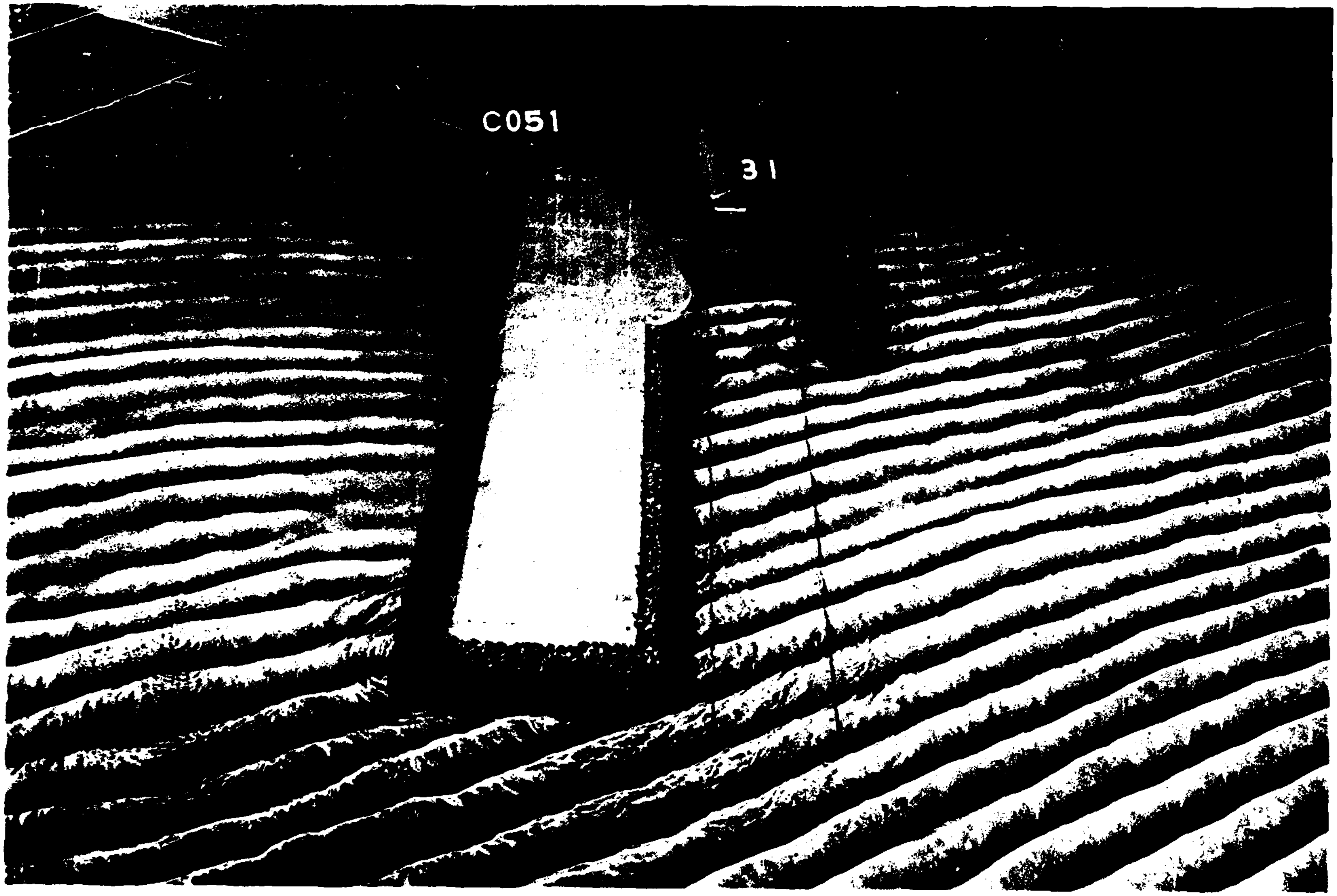

Photo 32. Typical wave patterns for Plan 4; 3.6-sec, 3.8-ft waves from 125 deg

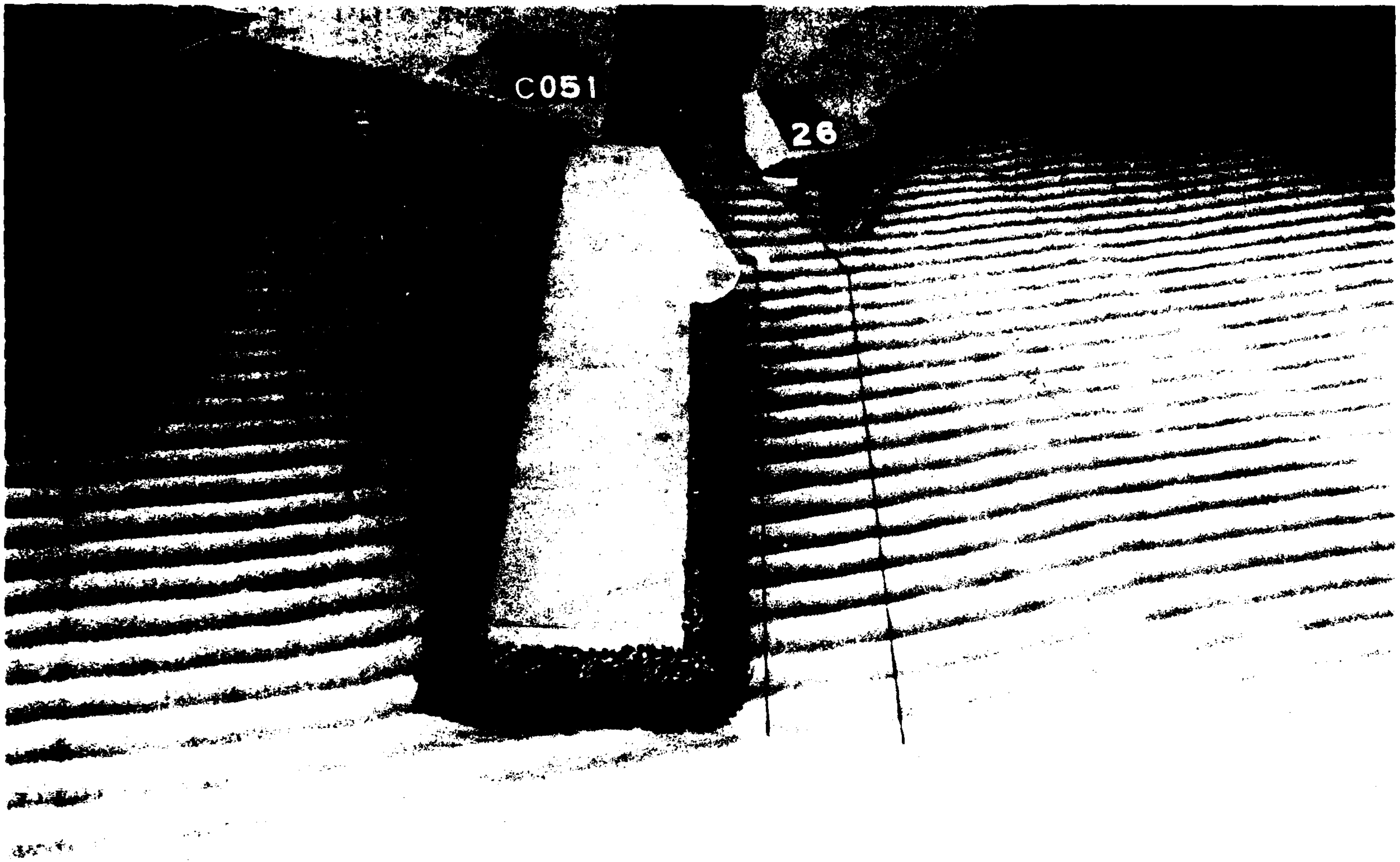

Photo 33. Typical wave patterns for Plan 5; 3.5-sec, 1.7-ft waves from 140 deg

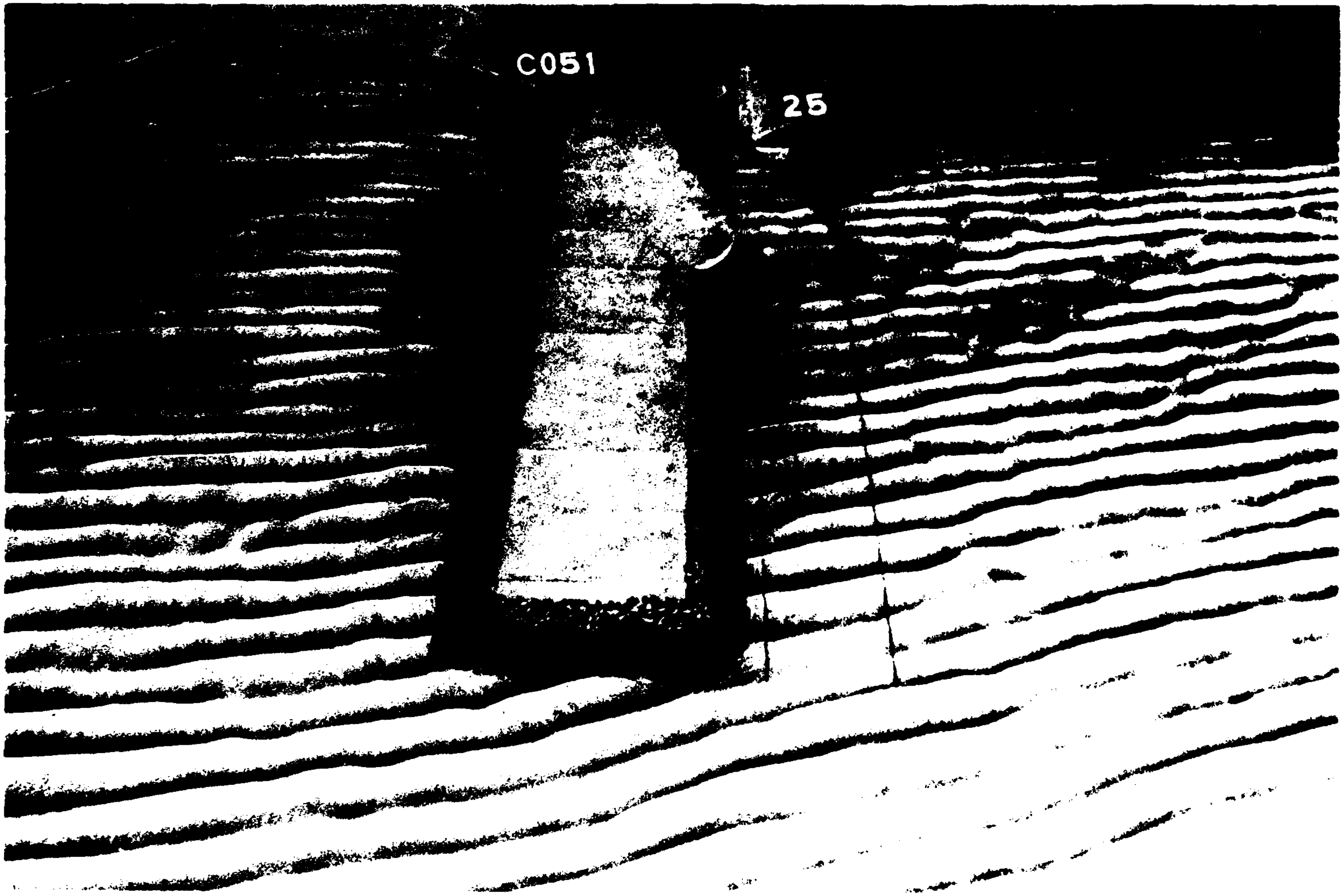

Photo 34. Typical wave patterns for Plan 5; 3.5-sec, 3.7-ft waves from 140 deg

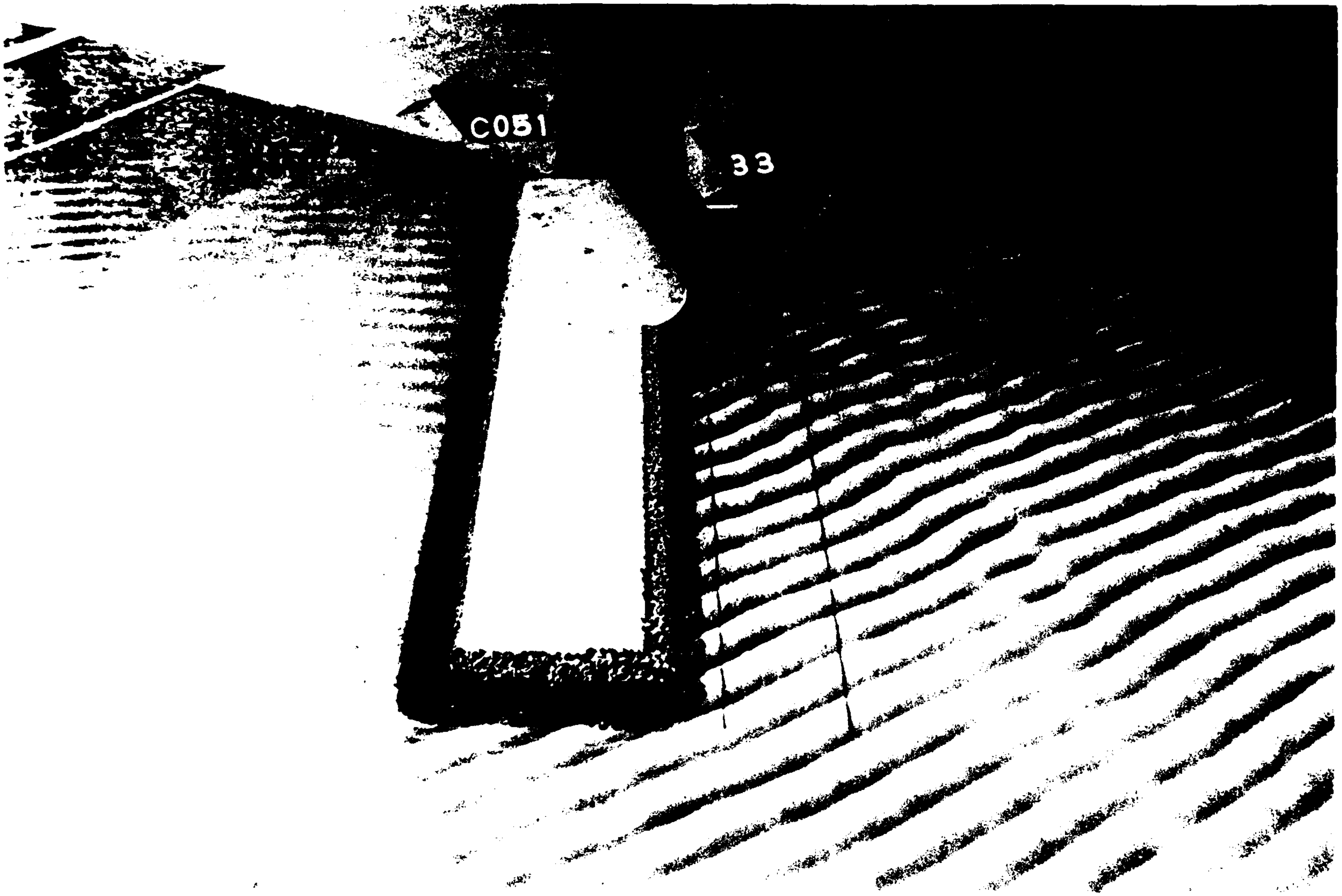

Photo 35. Typical wave patterns for Plan 5; 3.5-sec, 2.4-ft waves from 125 deg

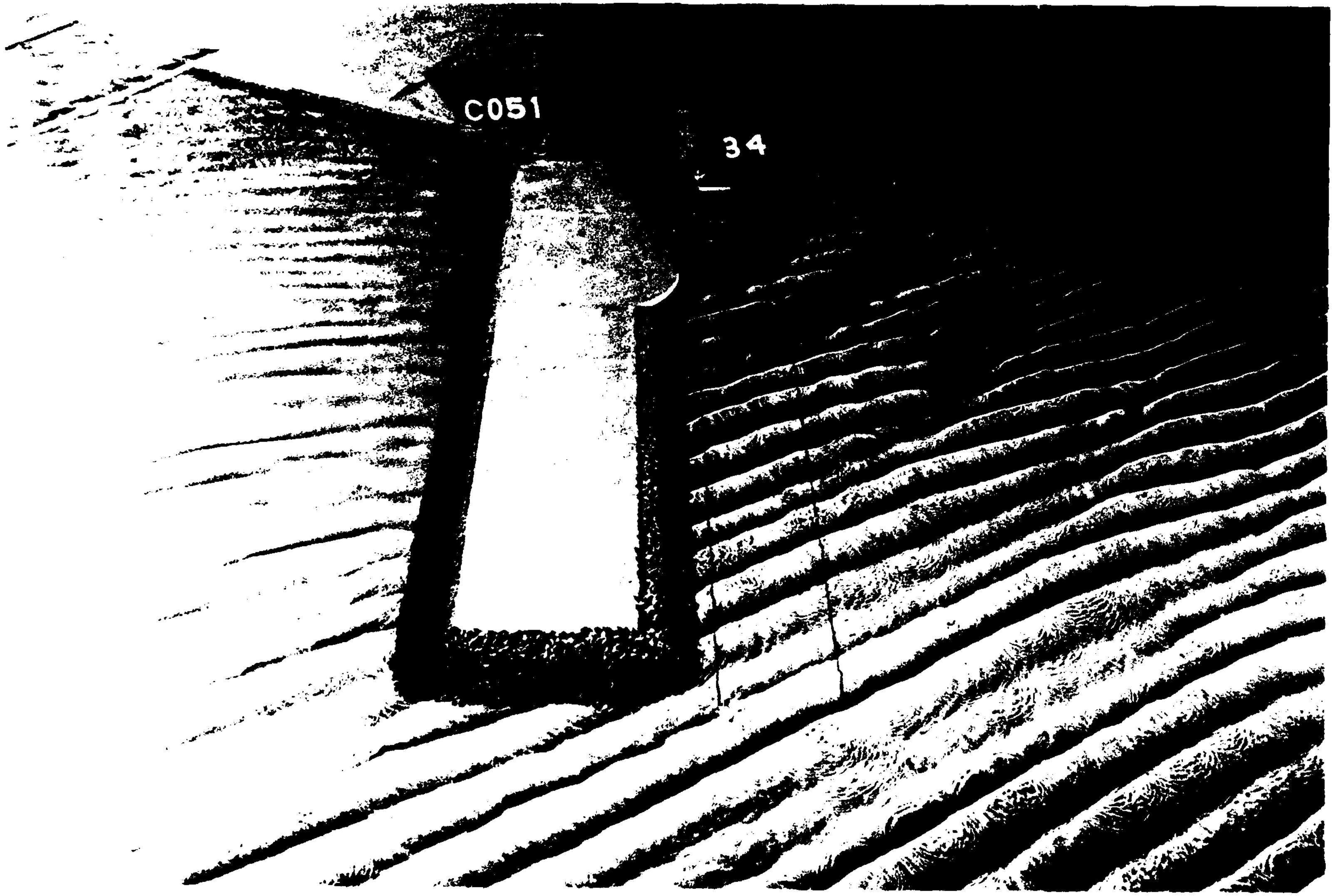

Photo 36. Typical wave patterns for Plan 5; 3.6-sec, 3.8-ft waves from 125 deg

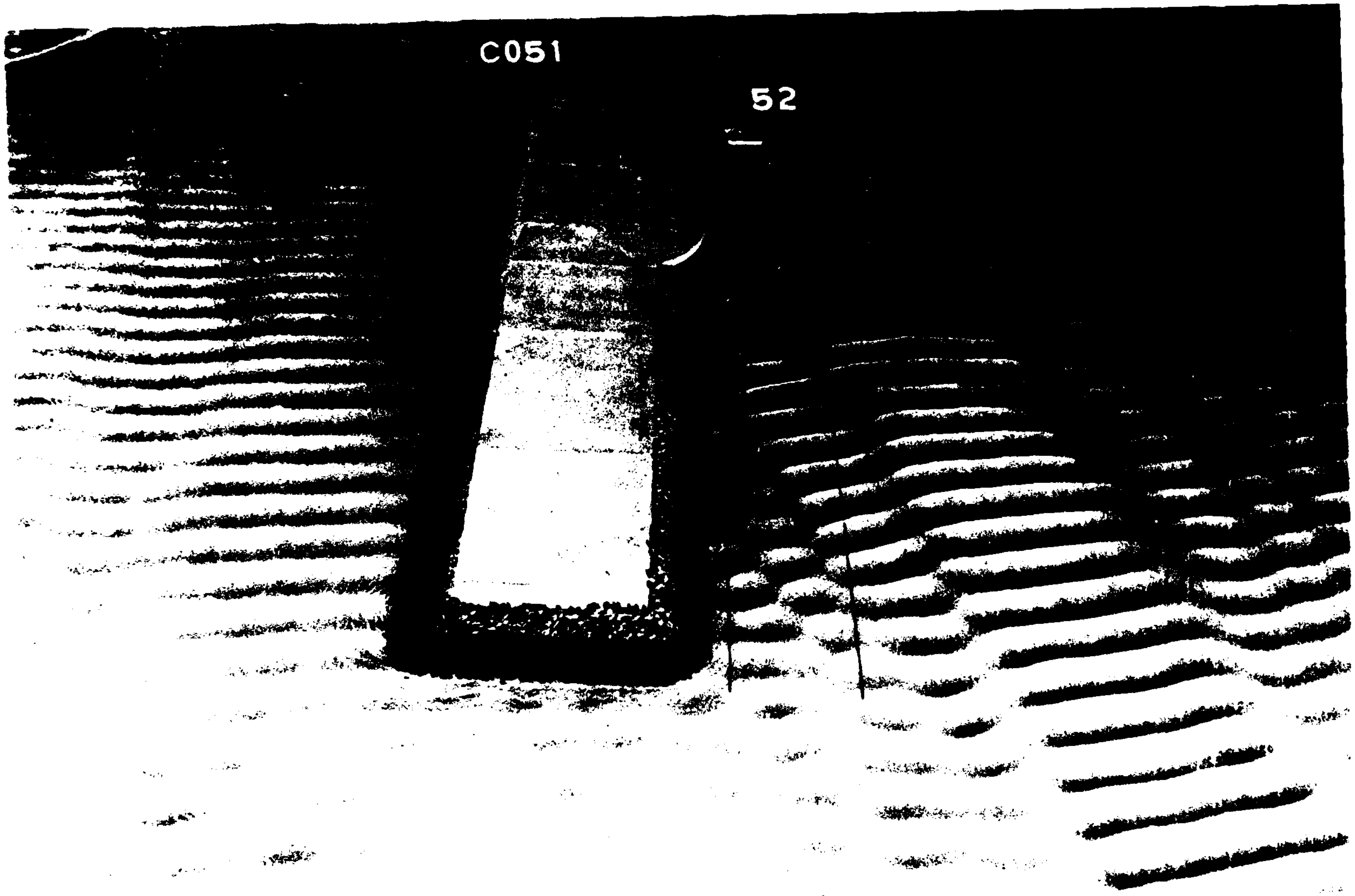

Photo 37. Typical wave patterns for Plan 6; 3.5-sec, 1.7-ft waves from 140 deg

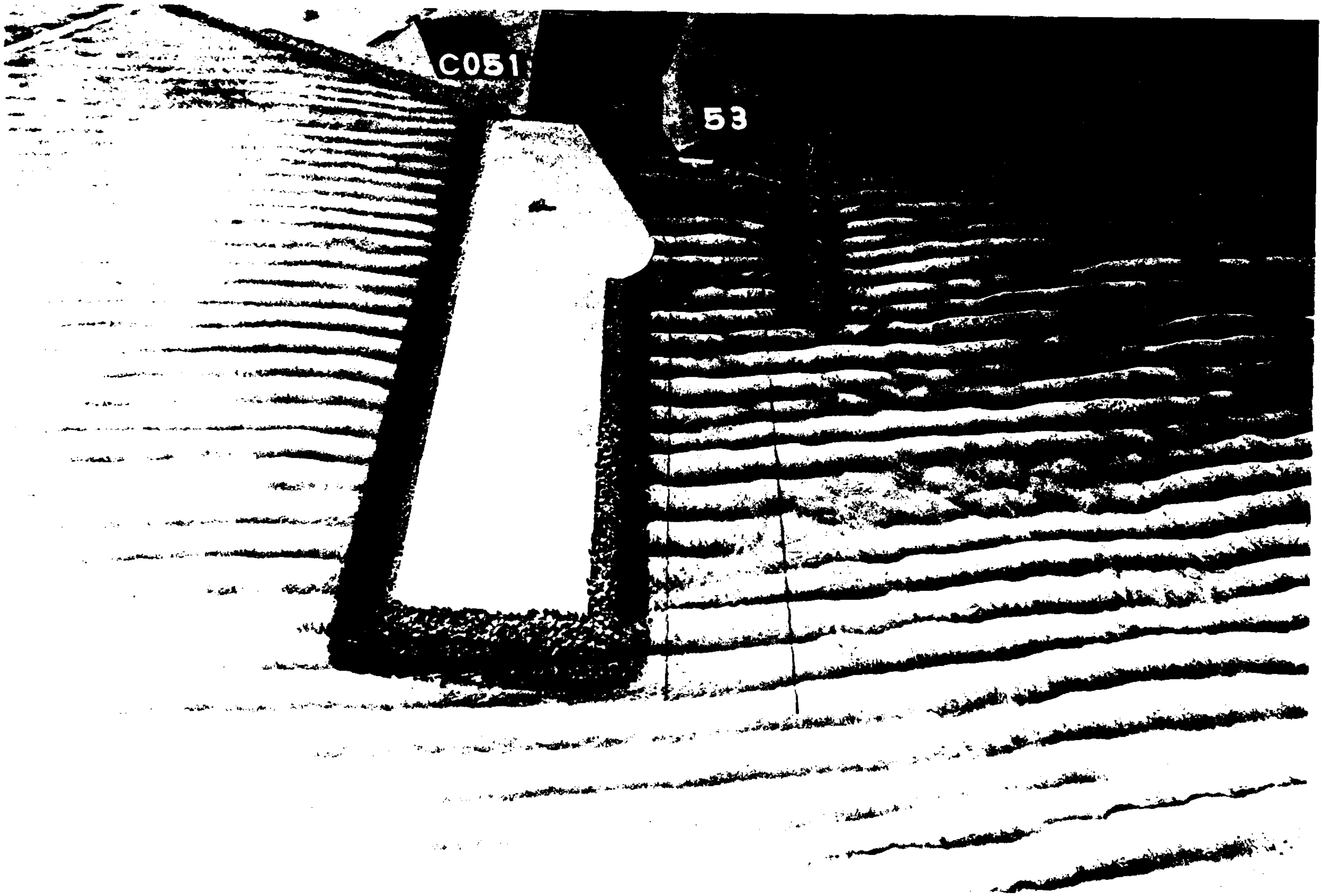

Photo 38. Typical wave patterns for Plan 6; 3.5-sec, 3.7-ft waves from 140 deg

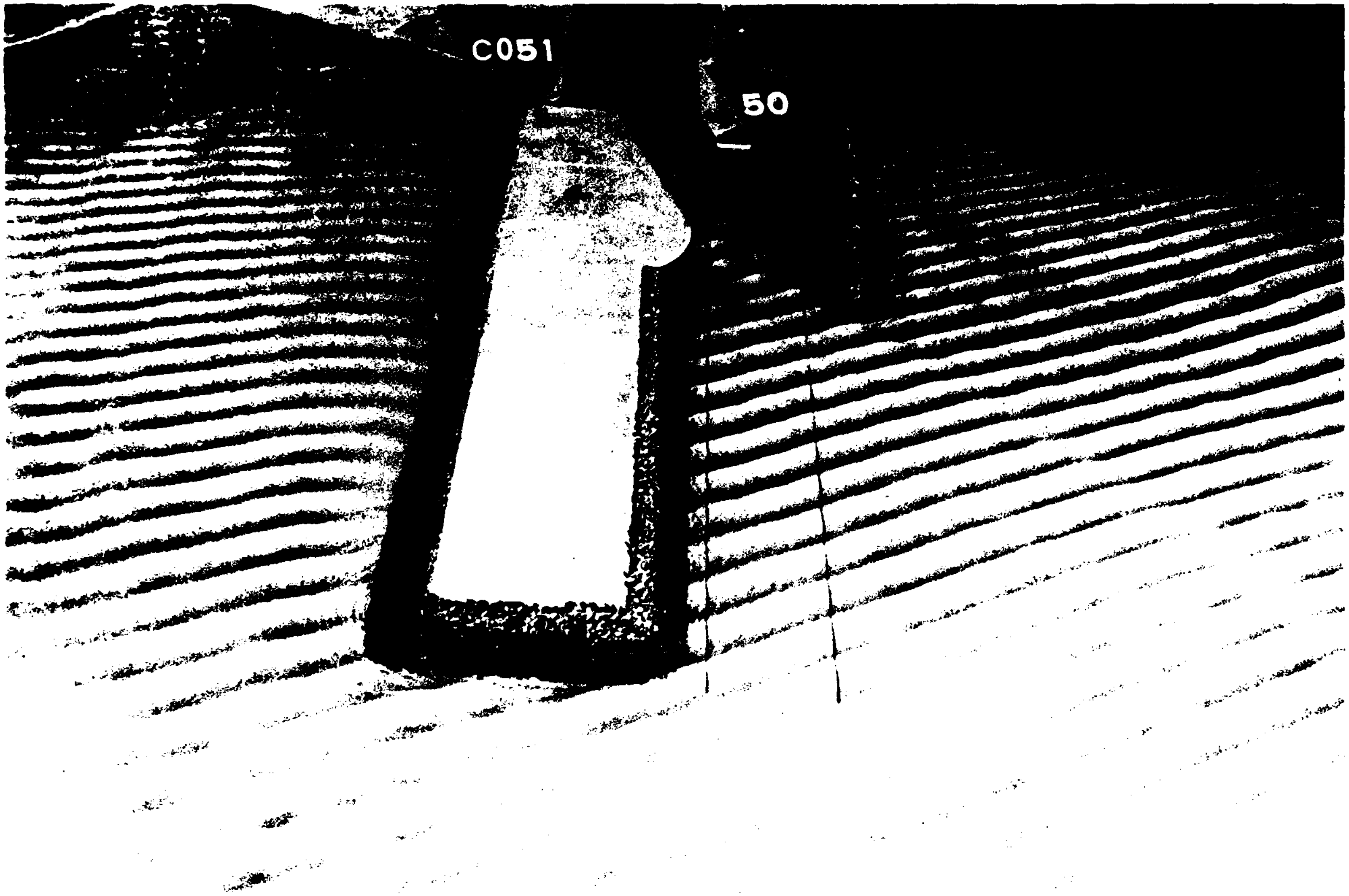

Photo 39. Typical wave patterns for Plan 6; 3.5-sec, 2.4-ft waves from 125 deg

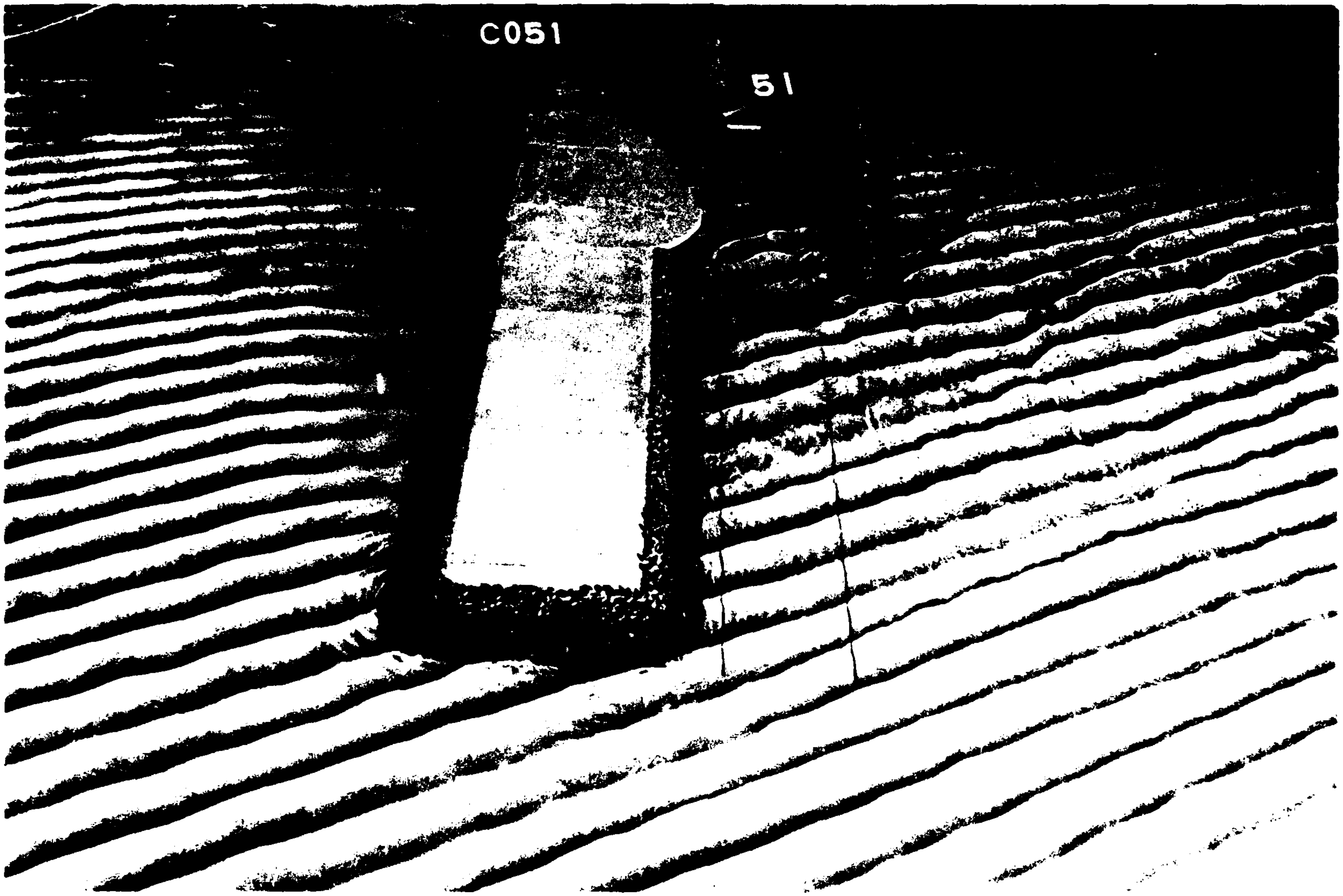

Photo 40. Typical wave patterns for Plan 6; 3.6-sec, 3.8-ft waves from 125 deg

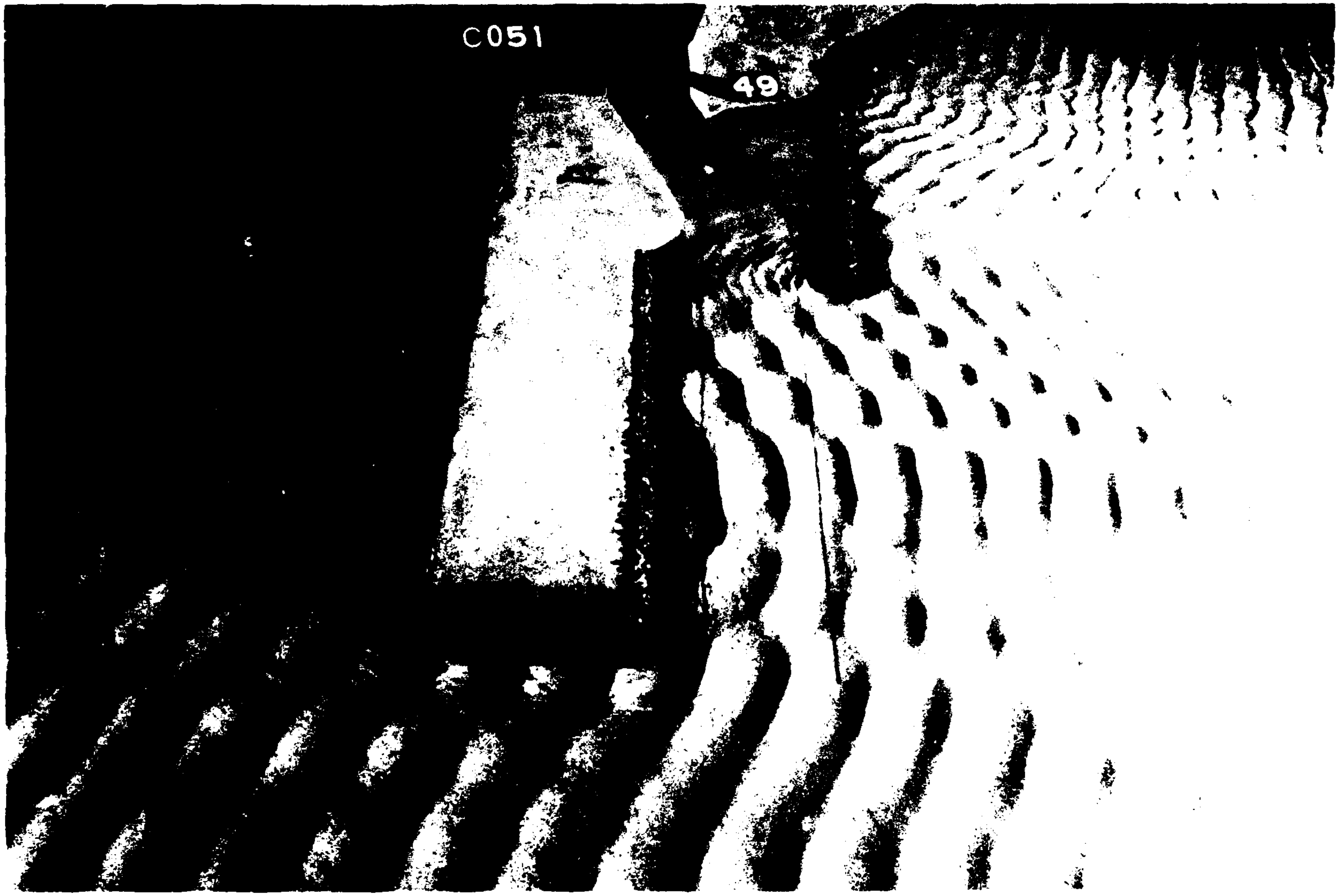

Photo 41. Typical wave patterns for Plan 6; 4.3-sec, 4.0-ft waves from 70 deg

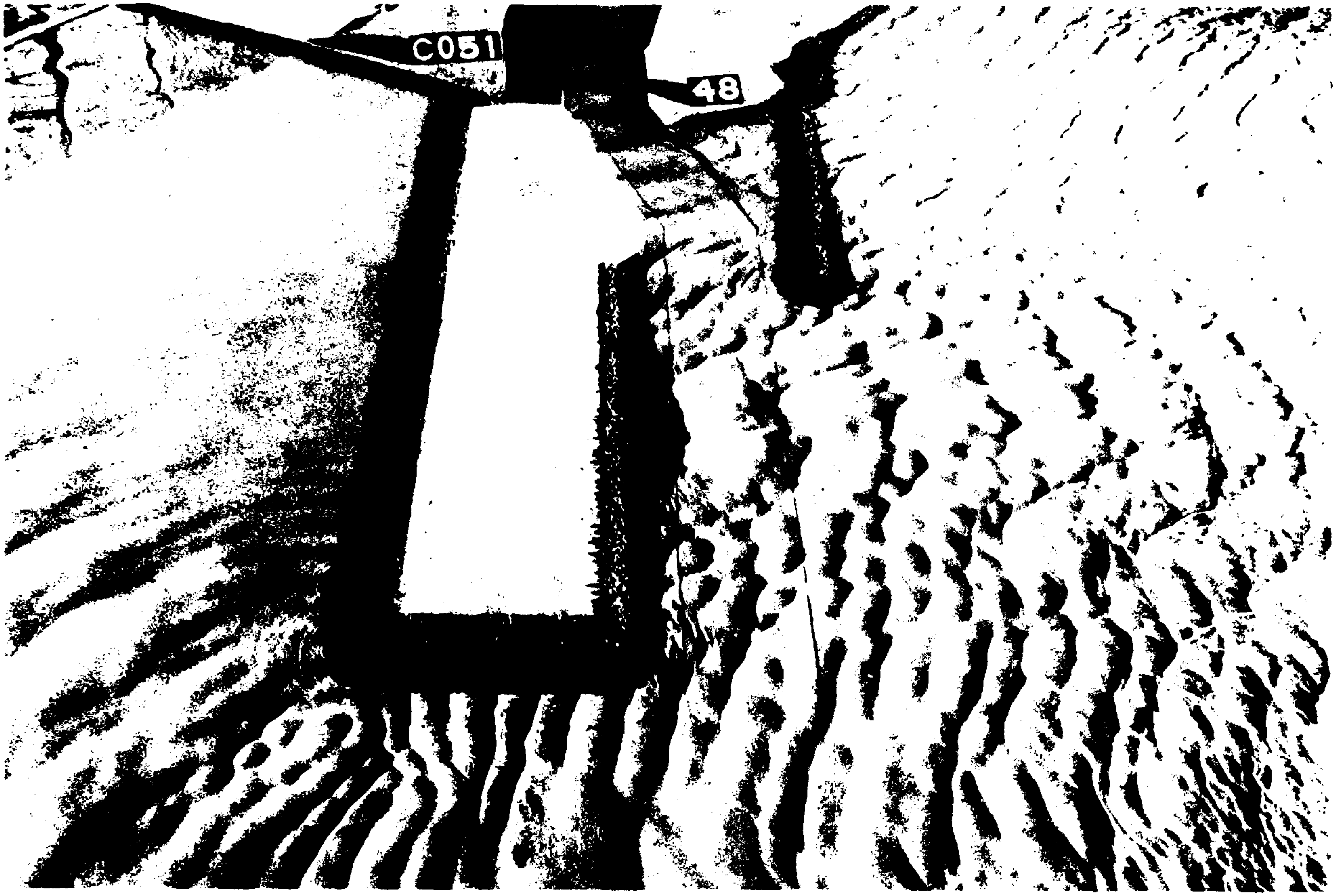

Photo 42. Typical wave patterns for Plan 6; 6.0-sec, 9.0-ft waves from 70 deg

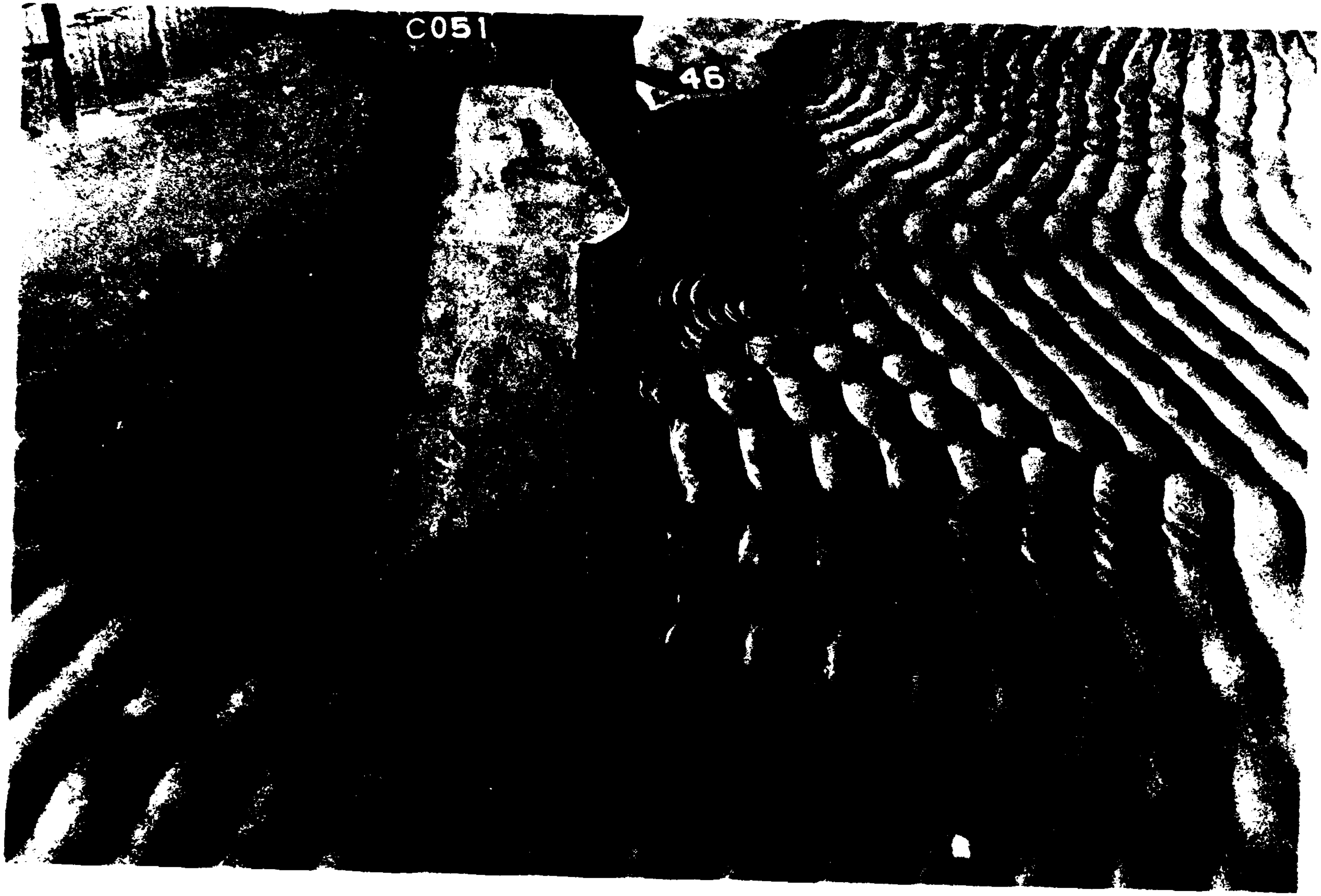

Photo 43. Typical wave patterns for Plan 7; 4.3-sec, 4.0-ft waves from 70 deg

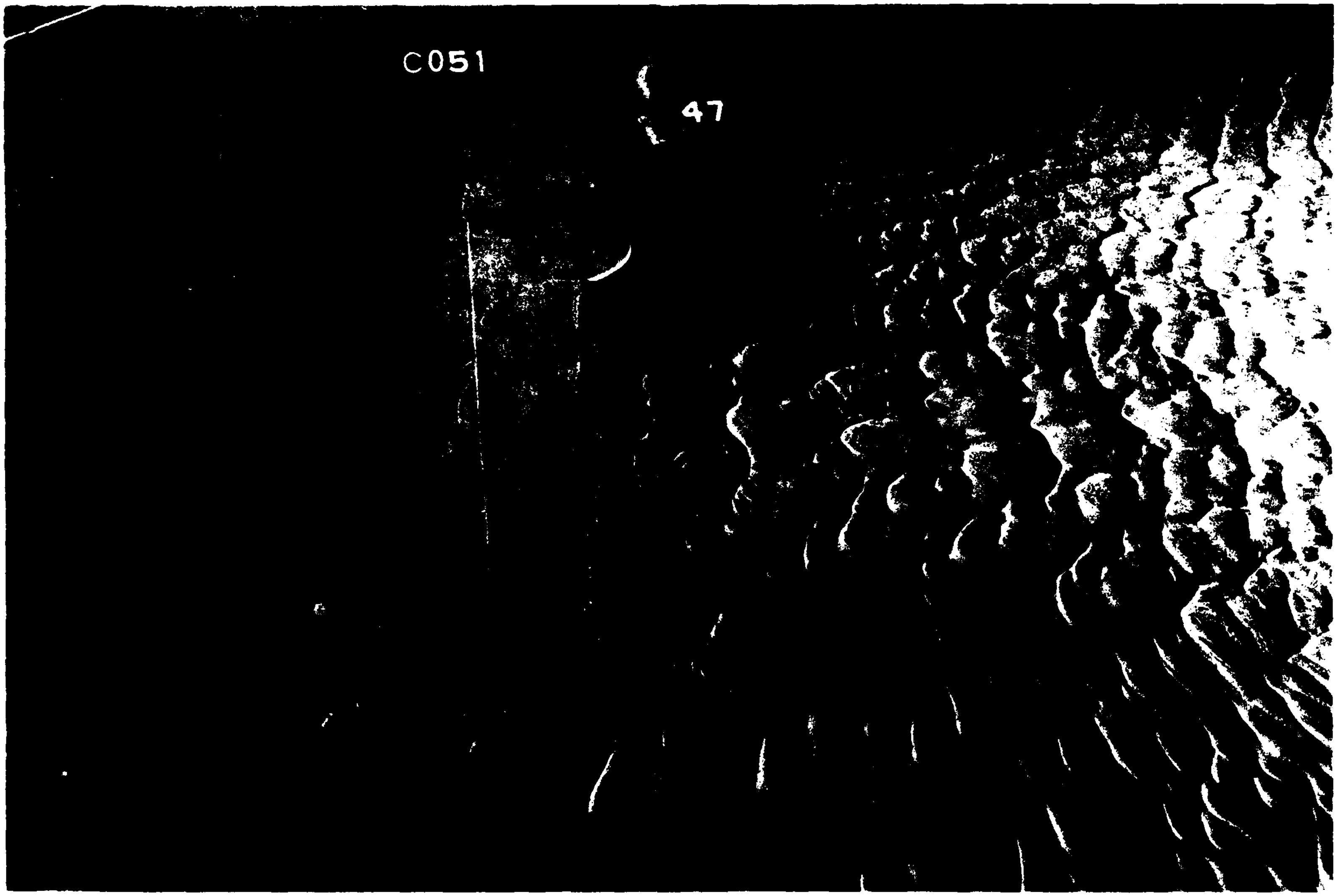

Photo 44. Typical wave patterns for Plan 7; 6.0-sec, 9.0-ft waves from 70 deg

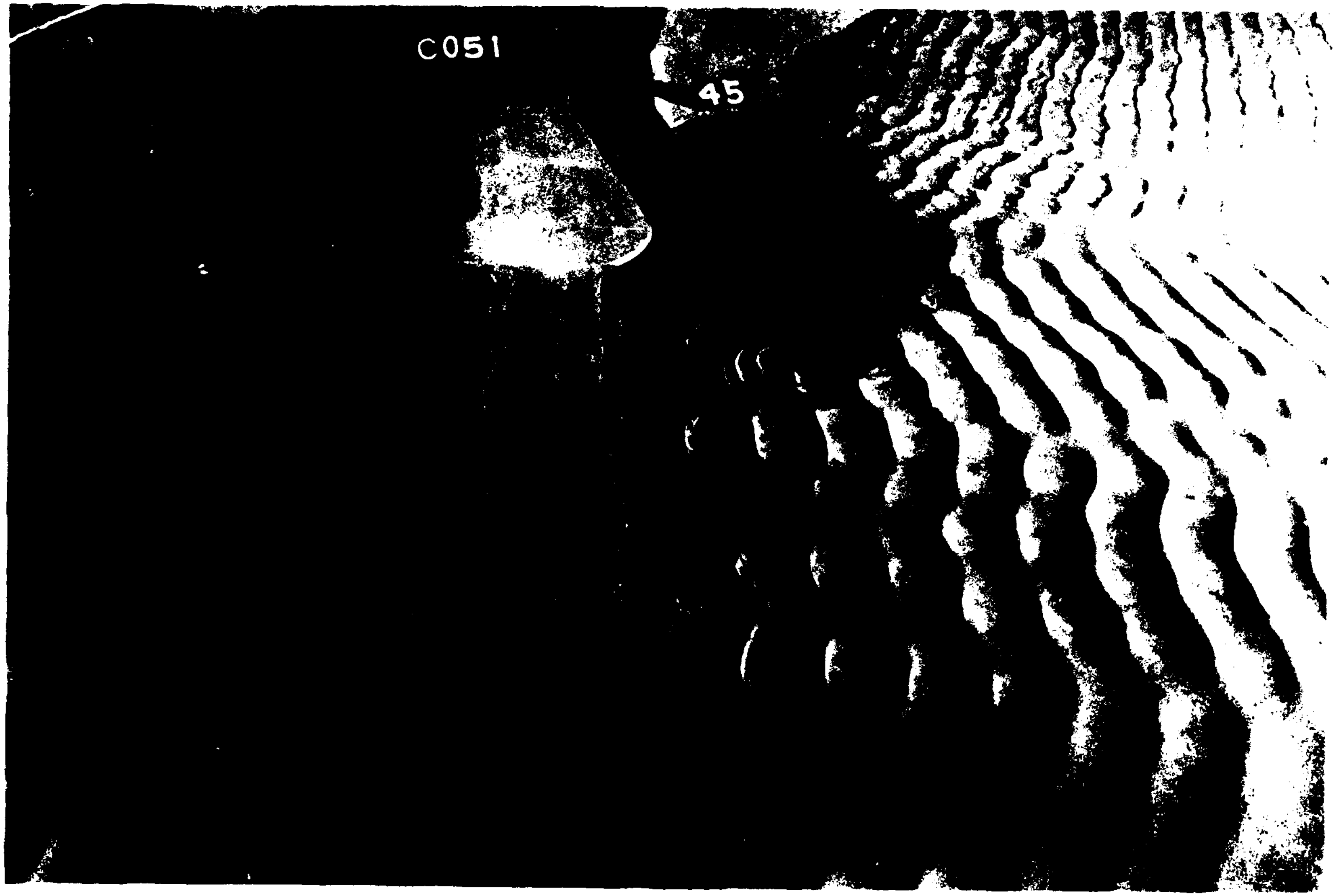

Photo 45. Typical wave patterns for Plan 8; 4.3-sec, 4.0-ft waves from 70 deg

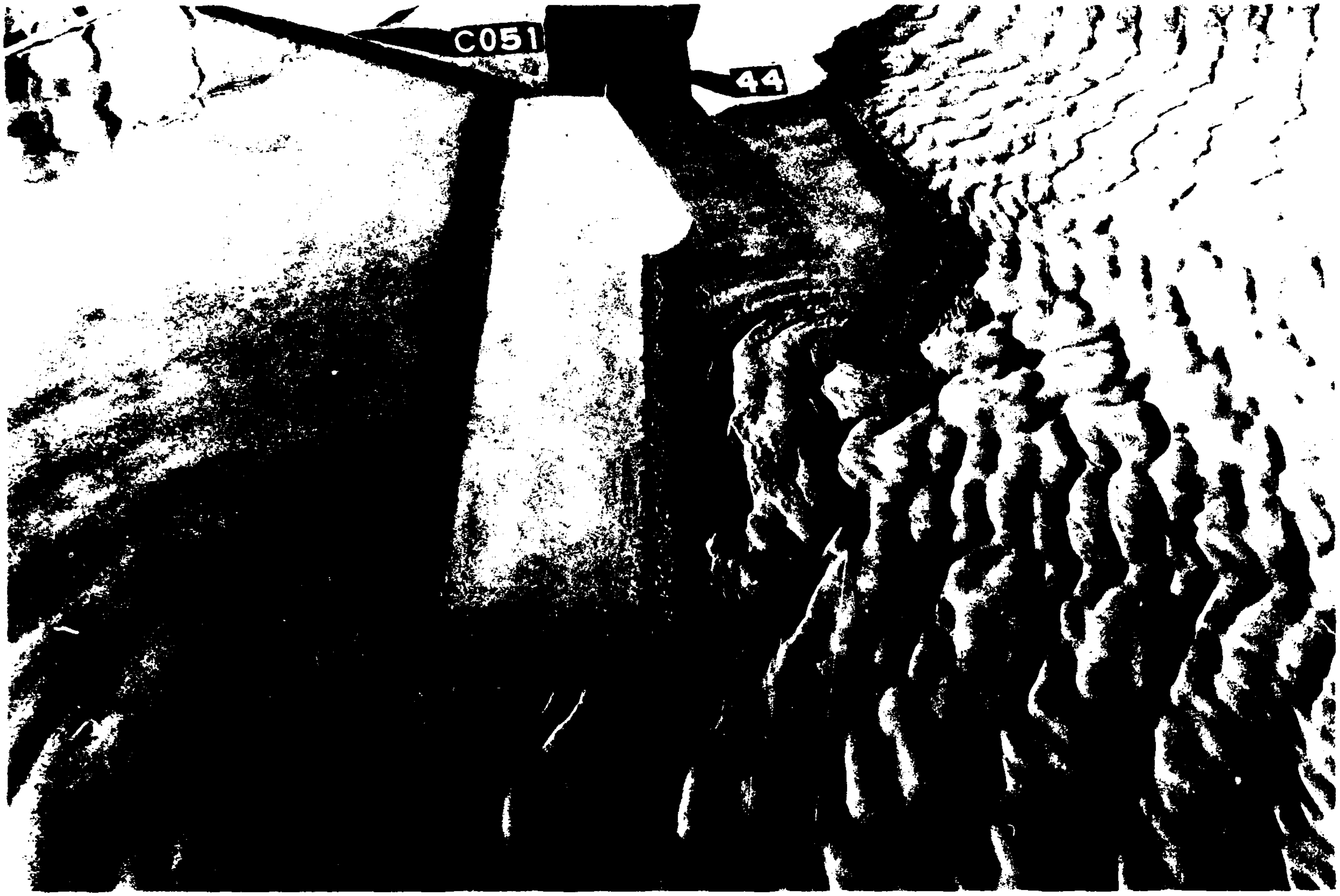

Photo 46. Typical wave patterns for Plan 8; 6.0-sec, 9.0-ft waves from 70 deg

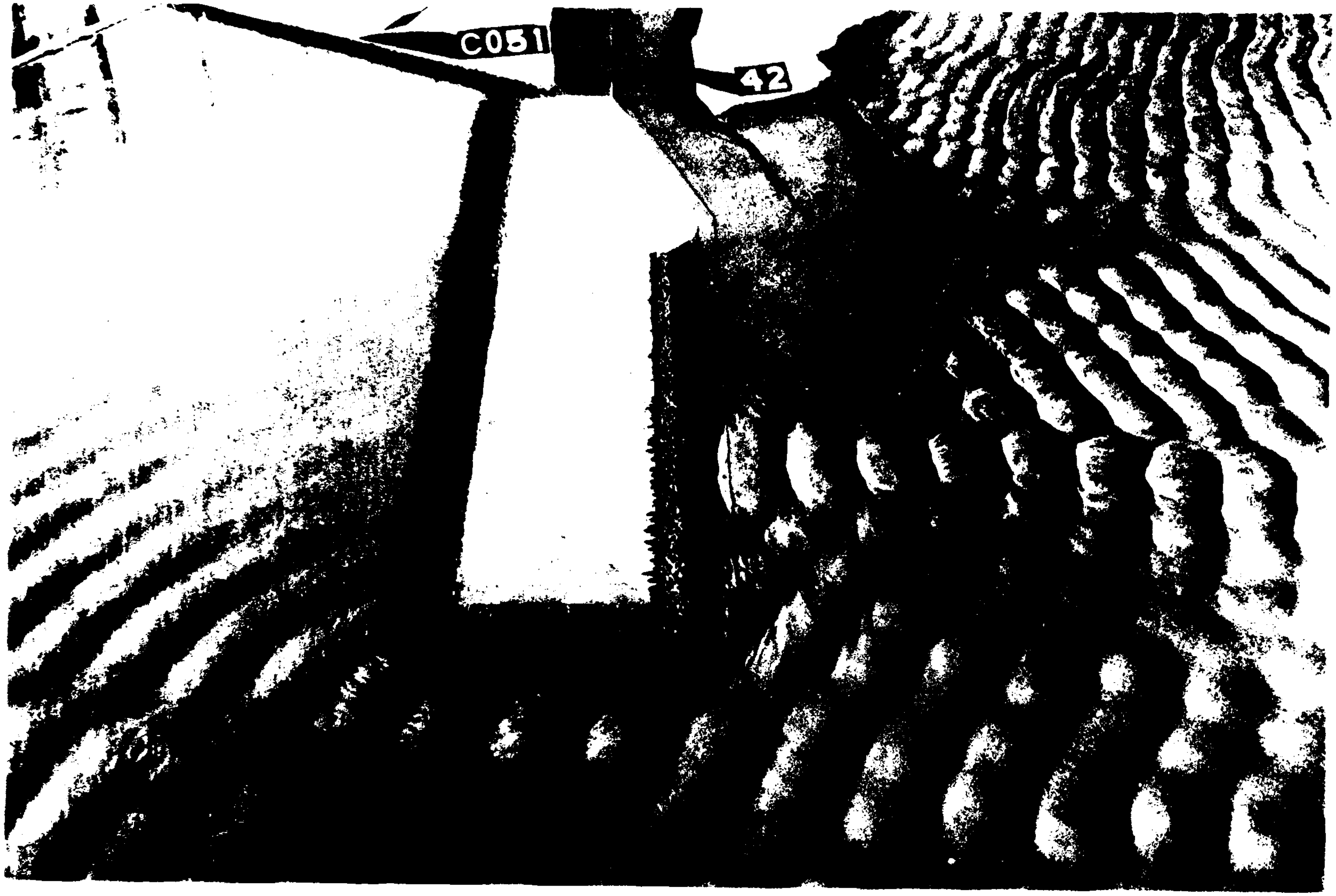

Photo 47. Typical wave patterns for Plan 9; 4.3-sec, 4.0-ft waves from 70 deg

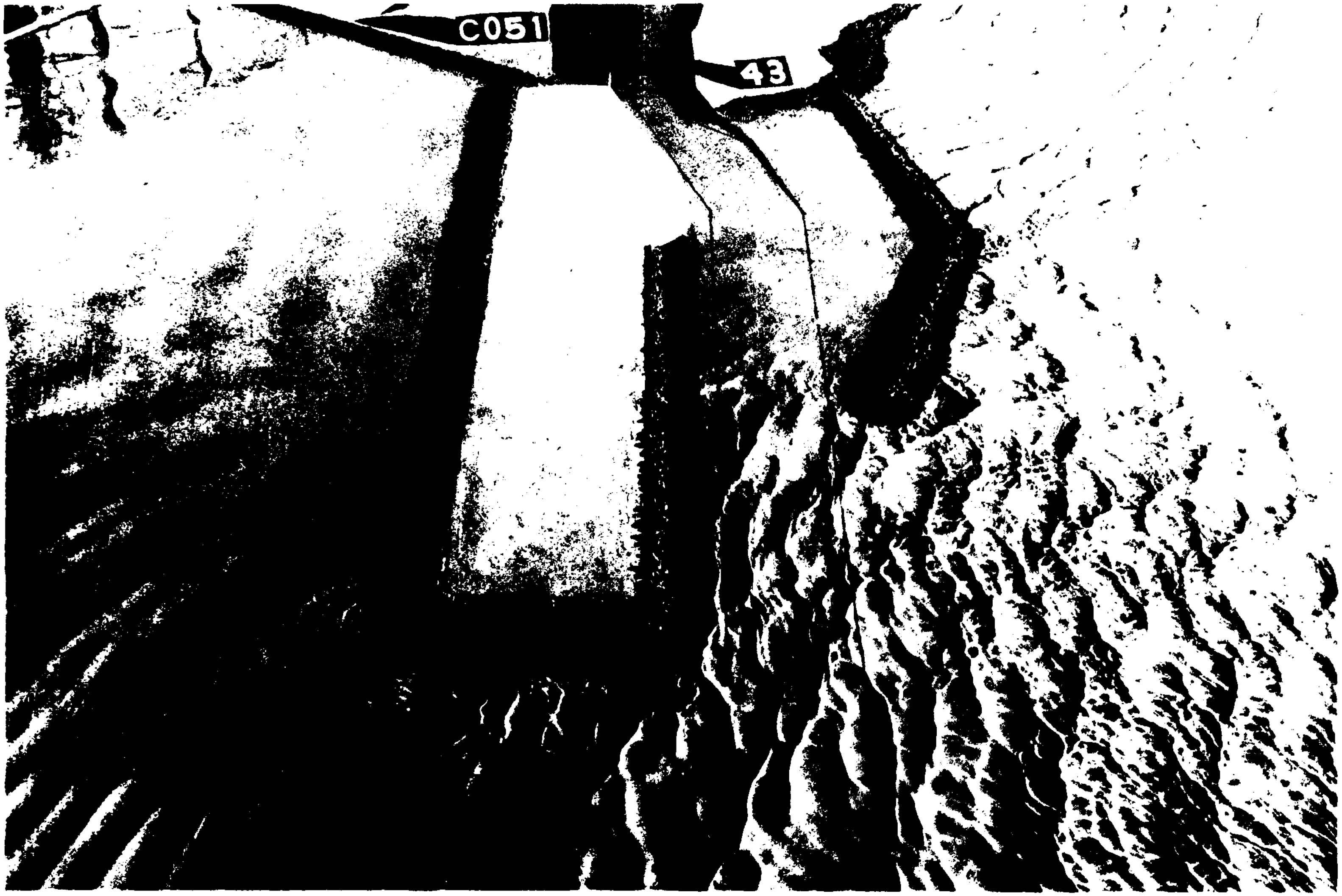

Photo 48. Typical wave patterns for Plan 9; 6.0-sec, 9.0-ft waves from 70 deg

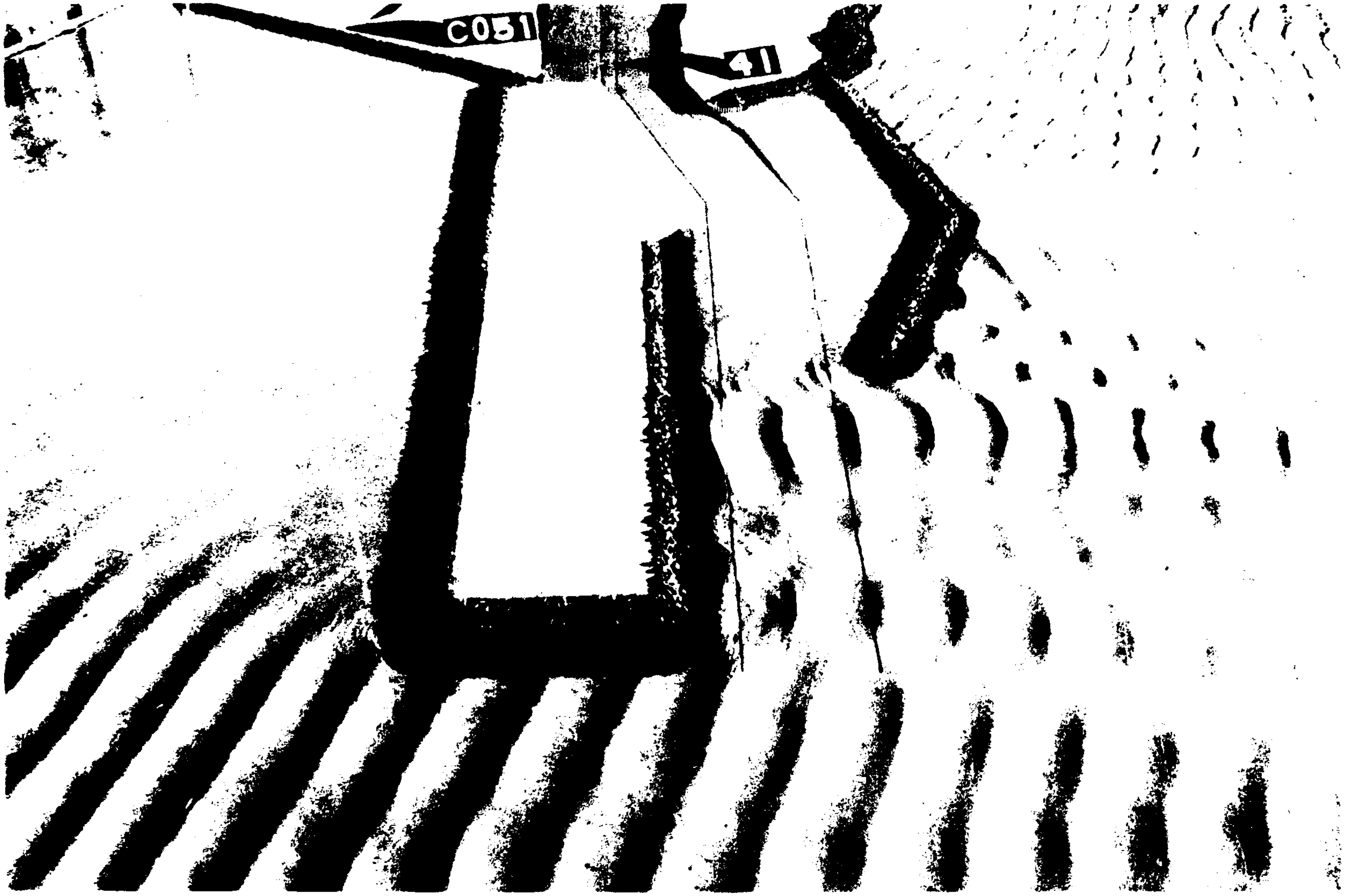

Photo 49. Typical wave patterns for Plan 10; 4.3-sec, 4.0-ft waves from 70 deg

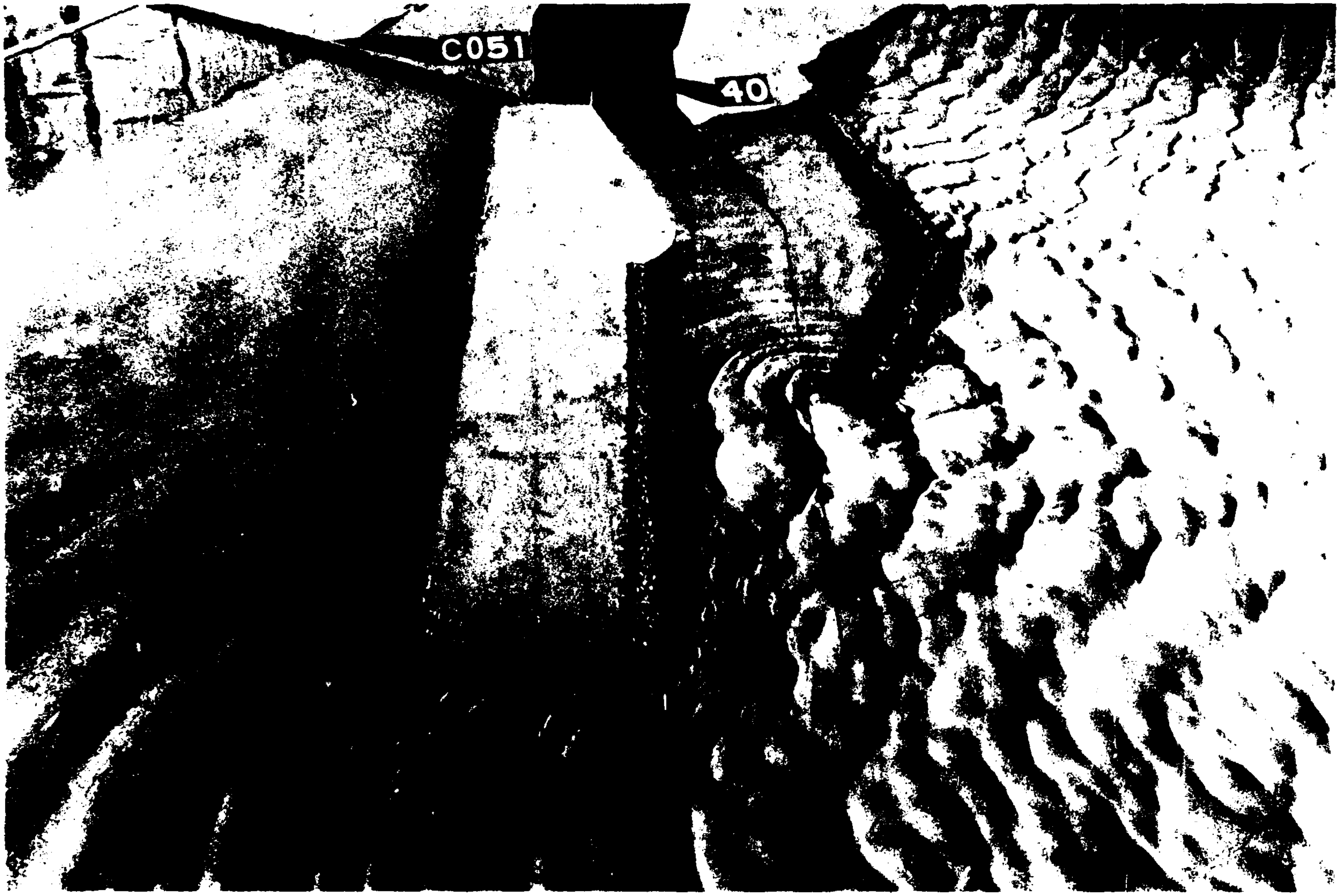

Photo 50. Typical wave patterns for Plan 10; 6.0-sec, 9.0-ft waves from 70 deg

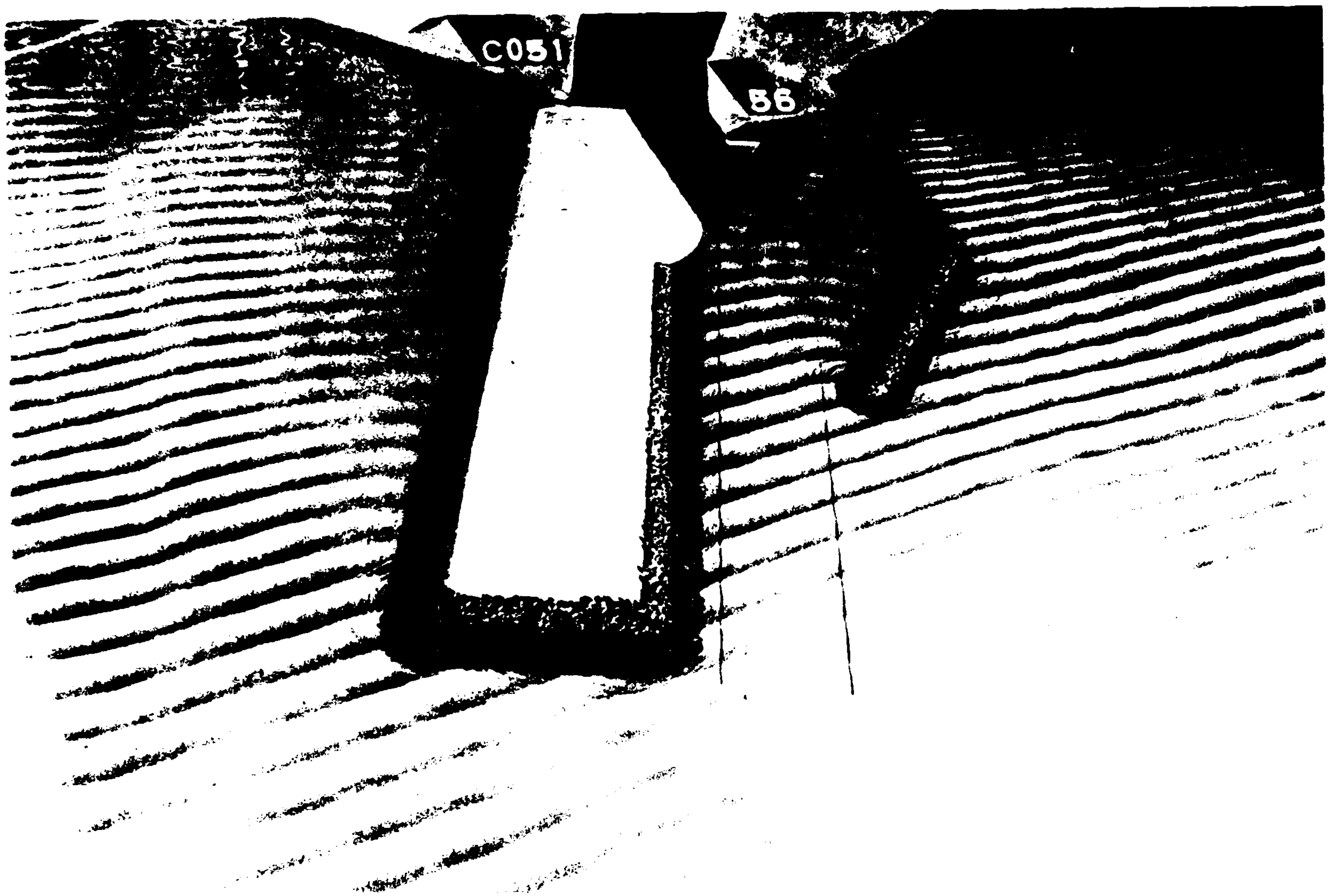

Photo 51. Typical wave patterns for Plan 10; 3.5-sec, 2.4-ft waves from 125 deg

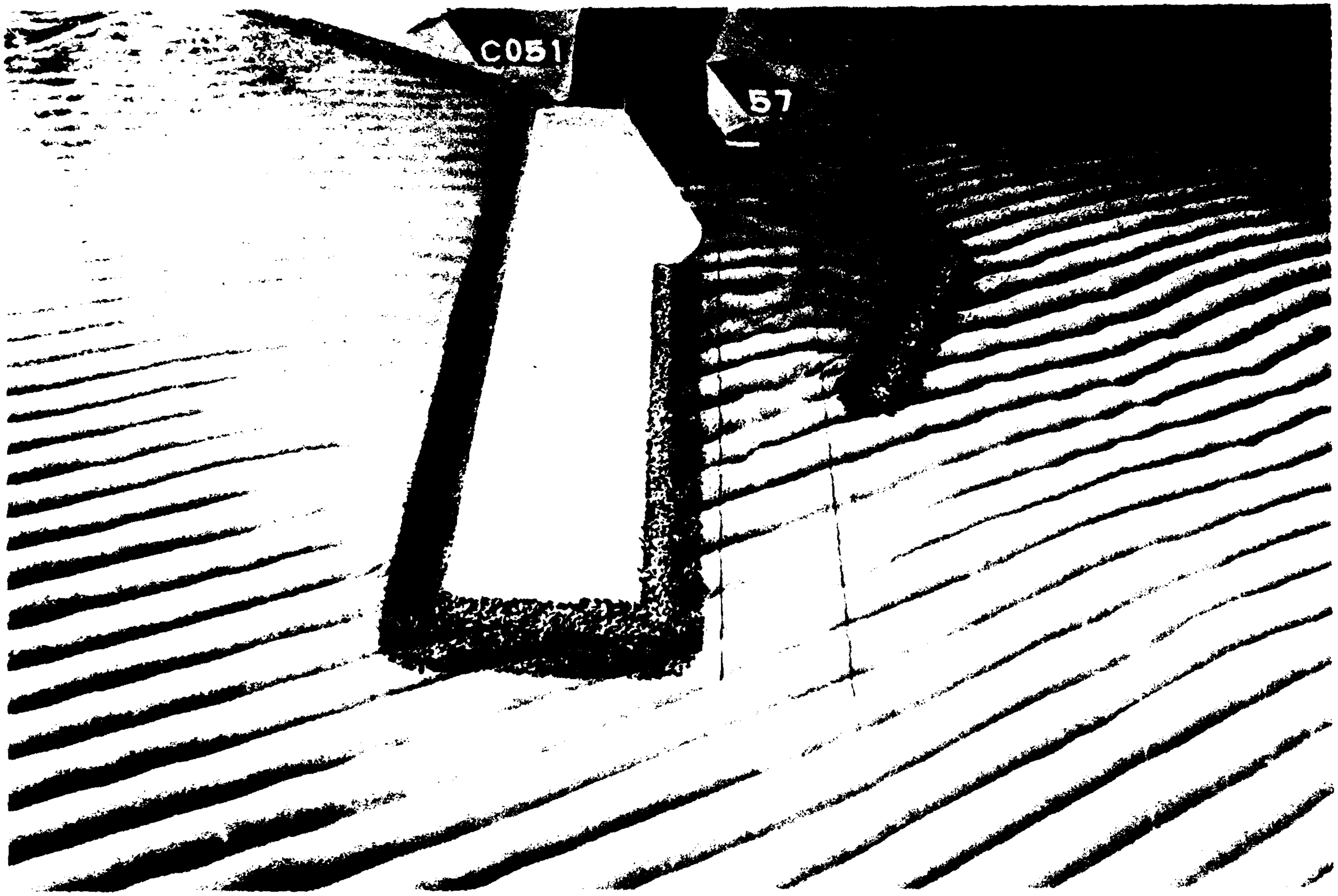

Photo 52. Typical wave patterns for Plan 10; 3.6-sec, 3.8-ft waves from 125 deg

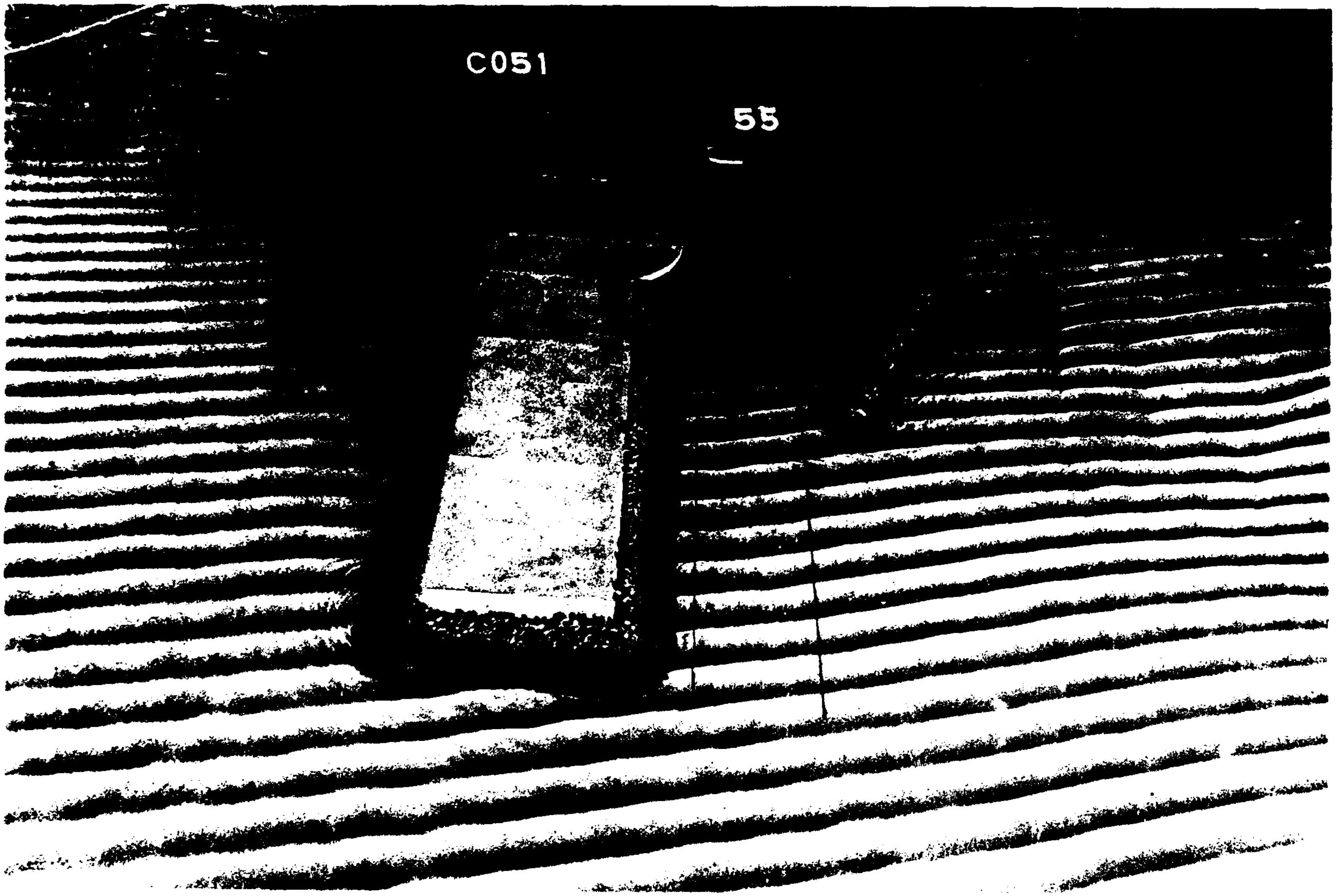

Photo 53. Typical wave patterns for Plan 10; 3.5-sec, 1.7-ft waves from 140 deg

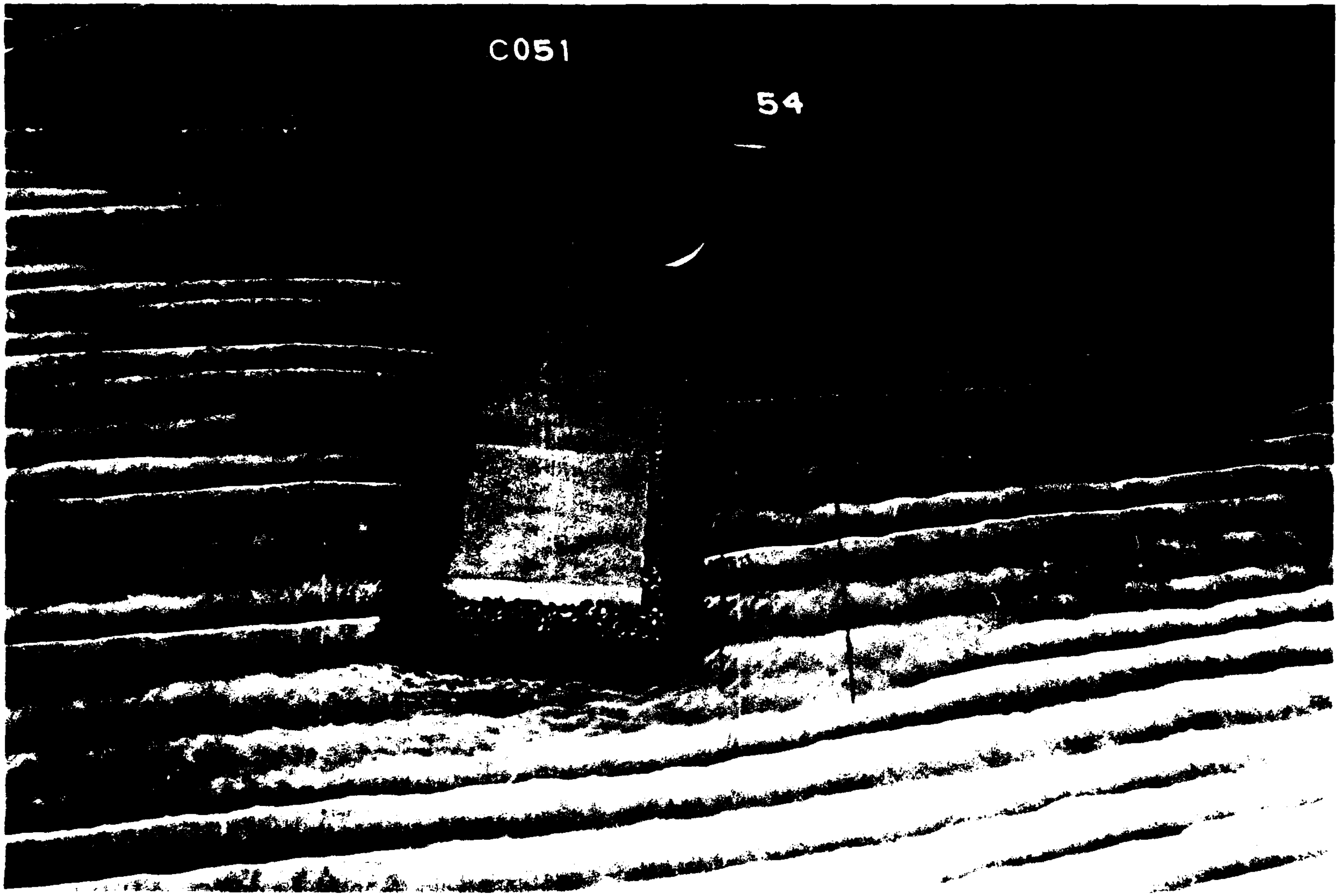

Photo 54. Typical wave patterns for Plan 10; 3.5-sec, 3.7-ft waves from 140 deg

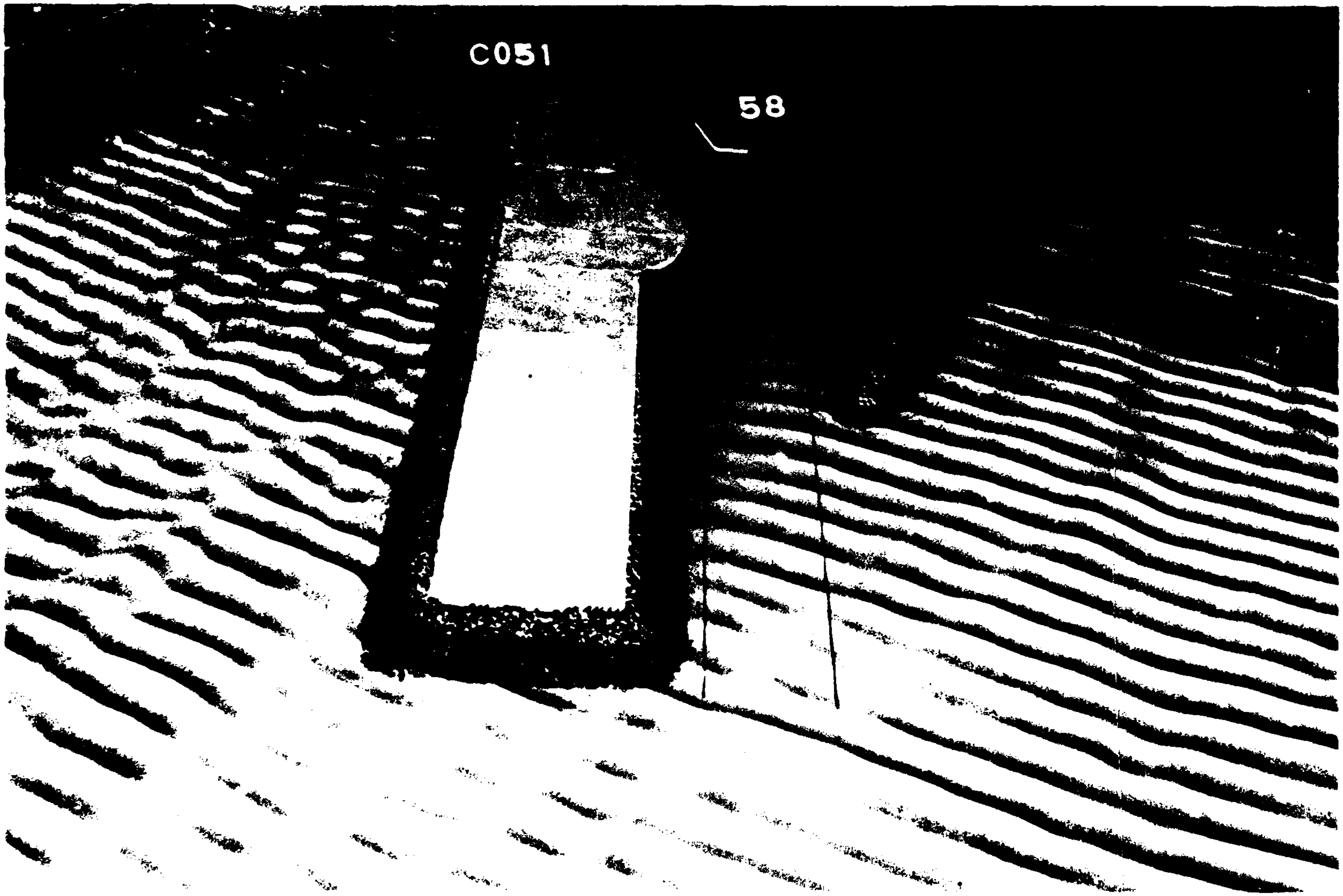

Photo 55. Typical wave patterns for Plan 10; 3.5-sec, 2.3-ft waves from 180 deg

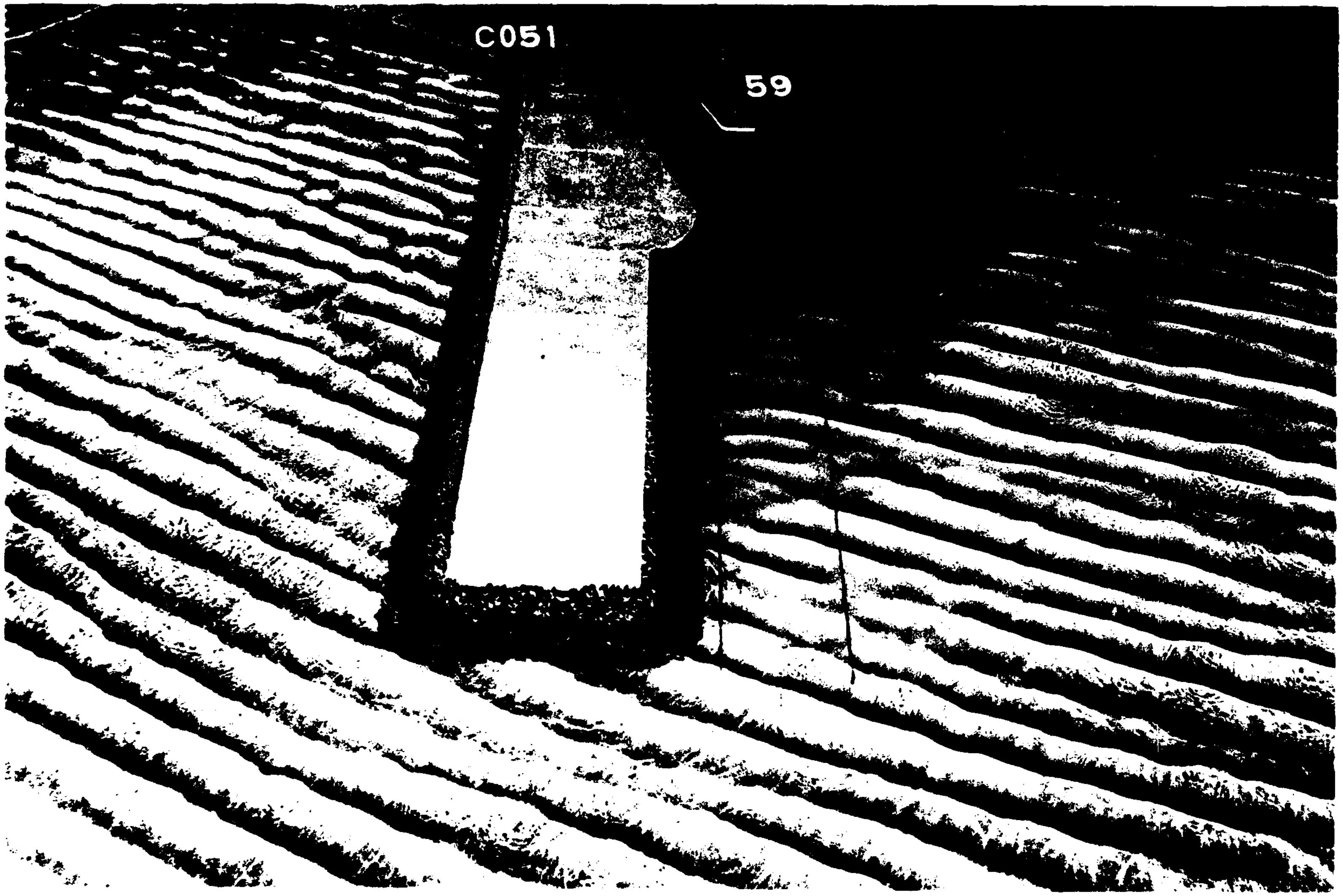

Photo 56. Typical wave patterns for Plan 10; 3.6-sec, 4.1-ft waves from 180 deg

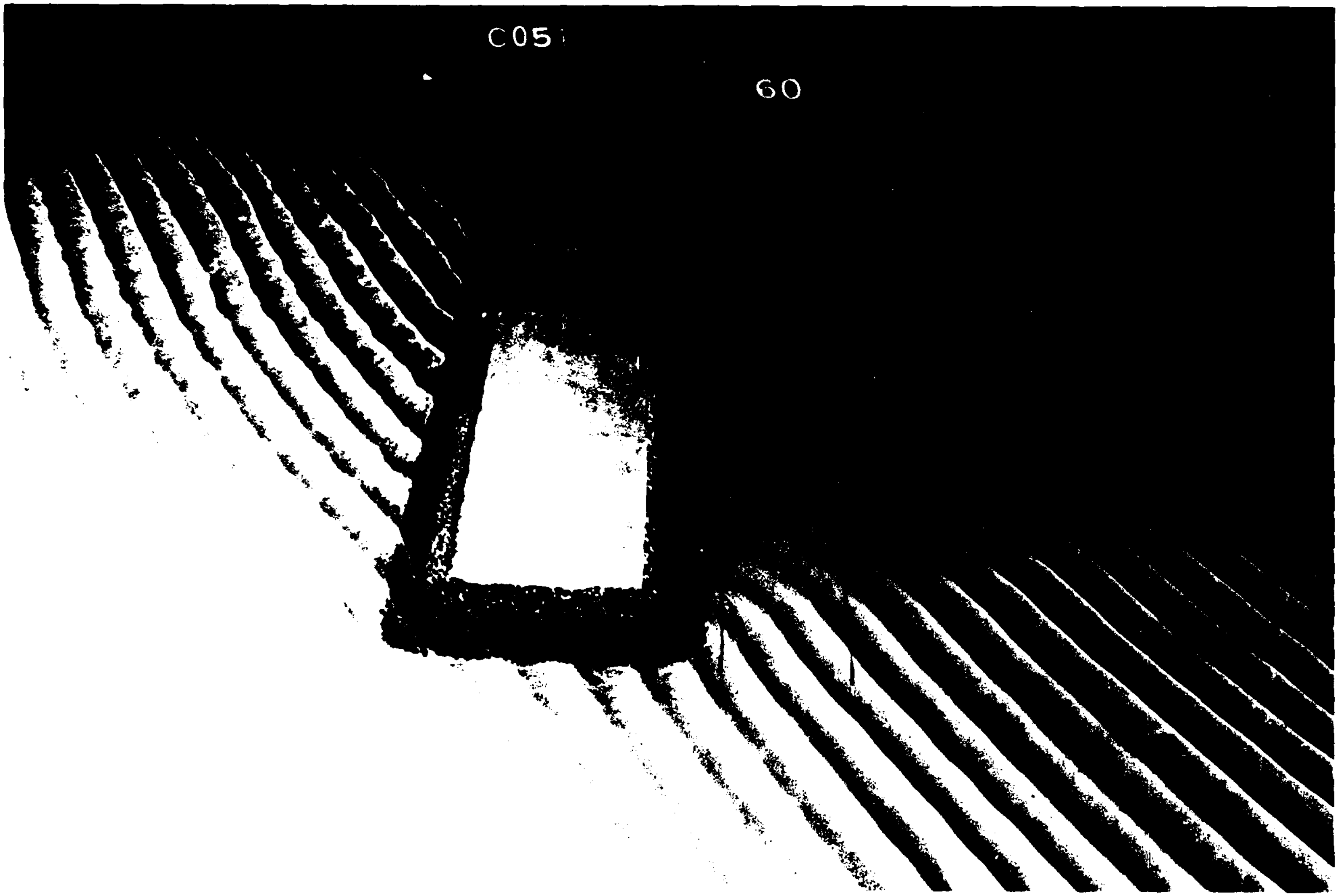

Photo 57. Typical wave patterns for Plan 10; 3.5-sec, 2.5-ft waves from 215 deg

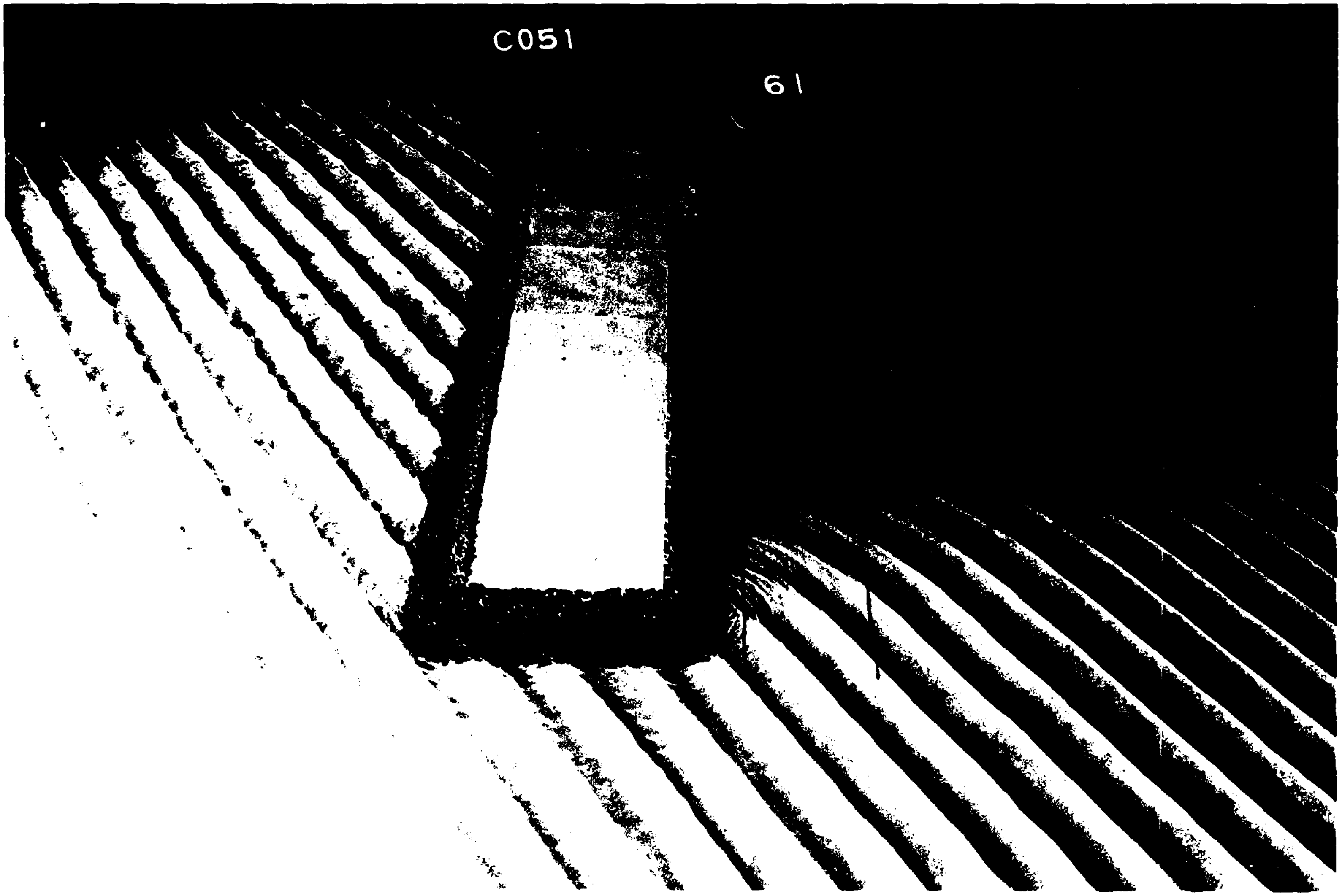

Photo 58. Typical wave patterns for Plan 10; 3.9-sec, 4.5-ft waves from 215 deg

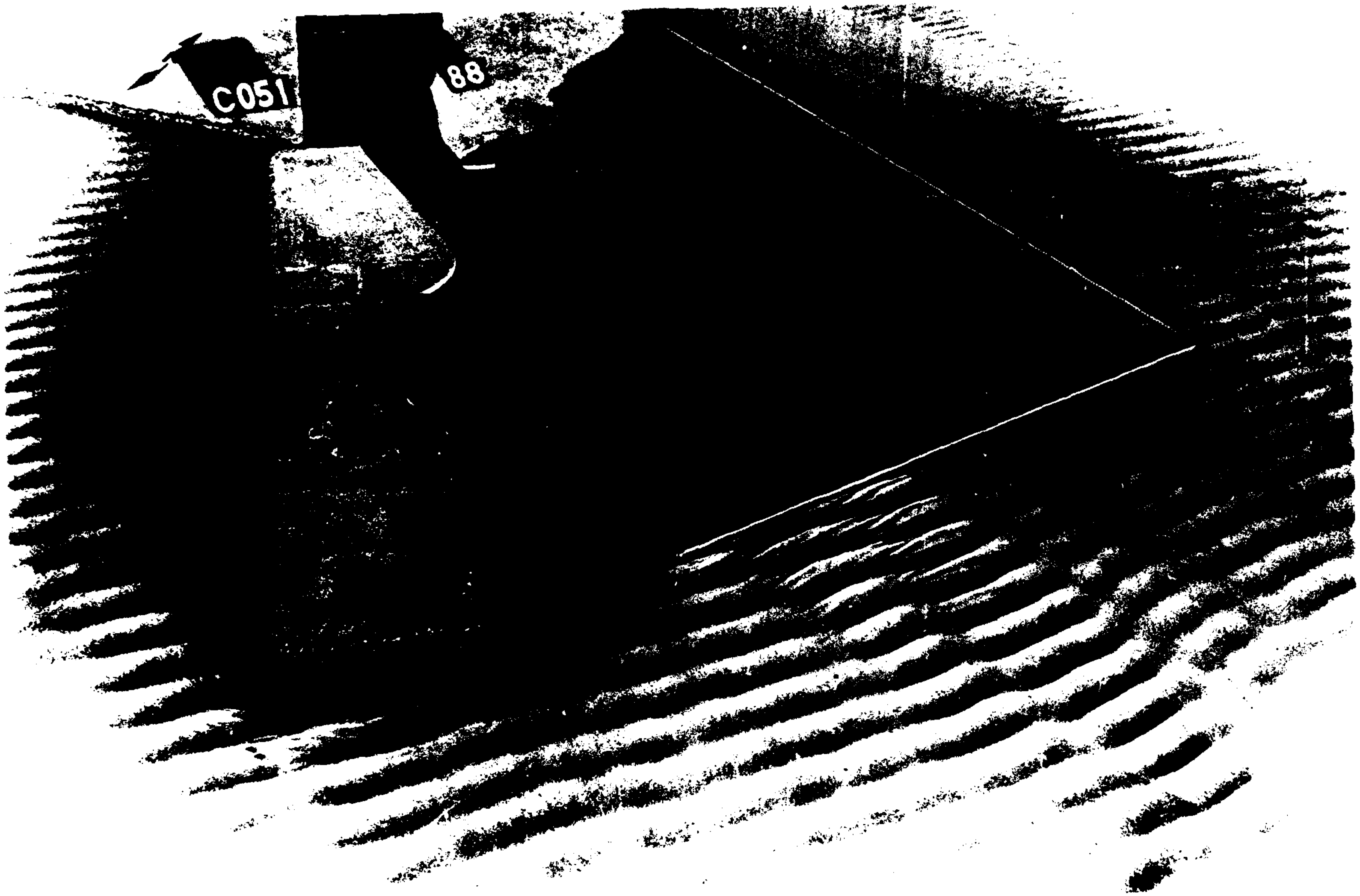

Photo 59. Typical wave patterns for Plan 11; 3.5-sec, 1.7-ft waves from 140 deg

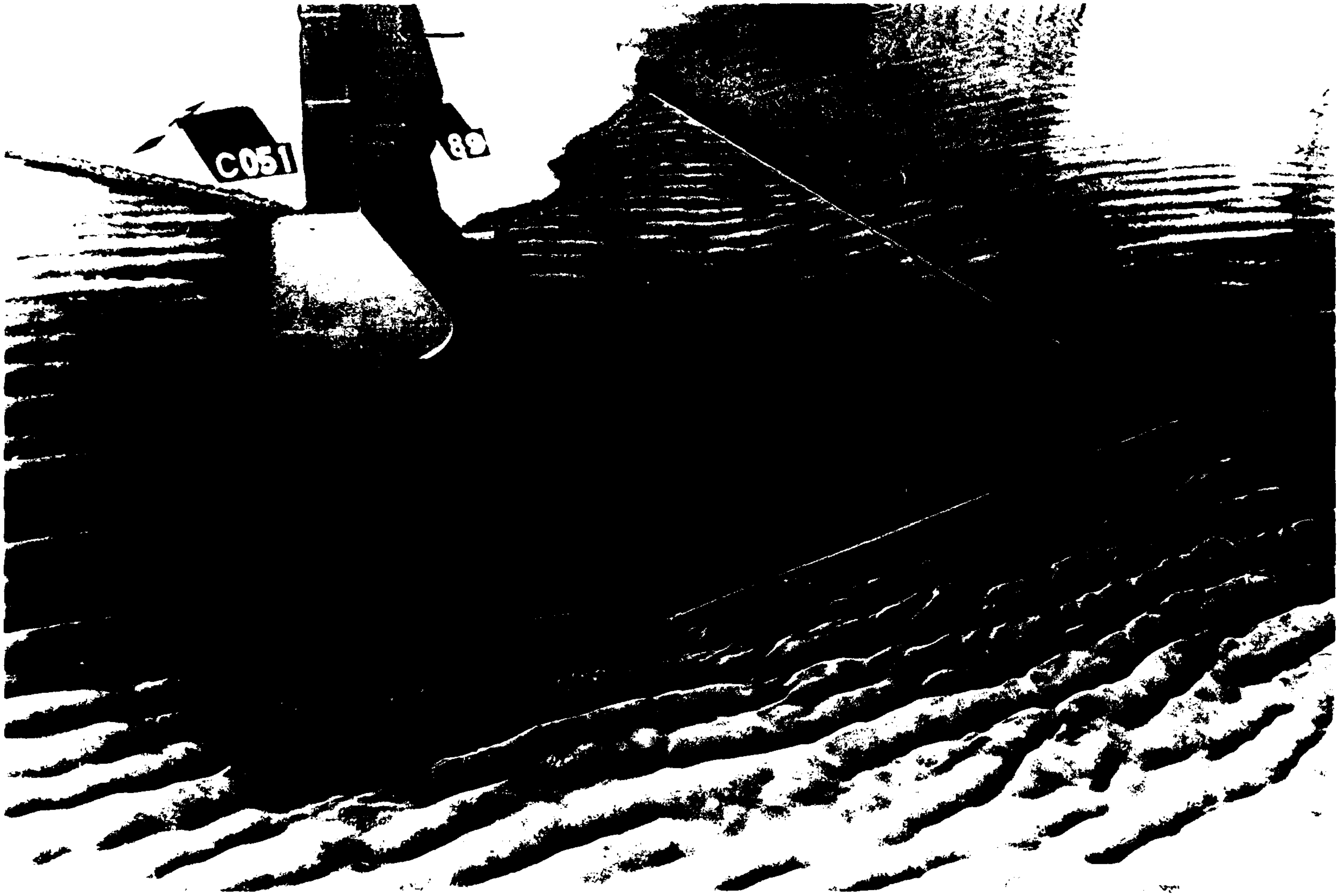

Photo 60. Typical wave patterns for Plan 11; 3.5-sec, 3.7-ft waves from 140 deg

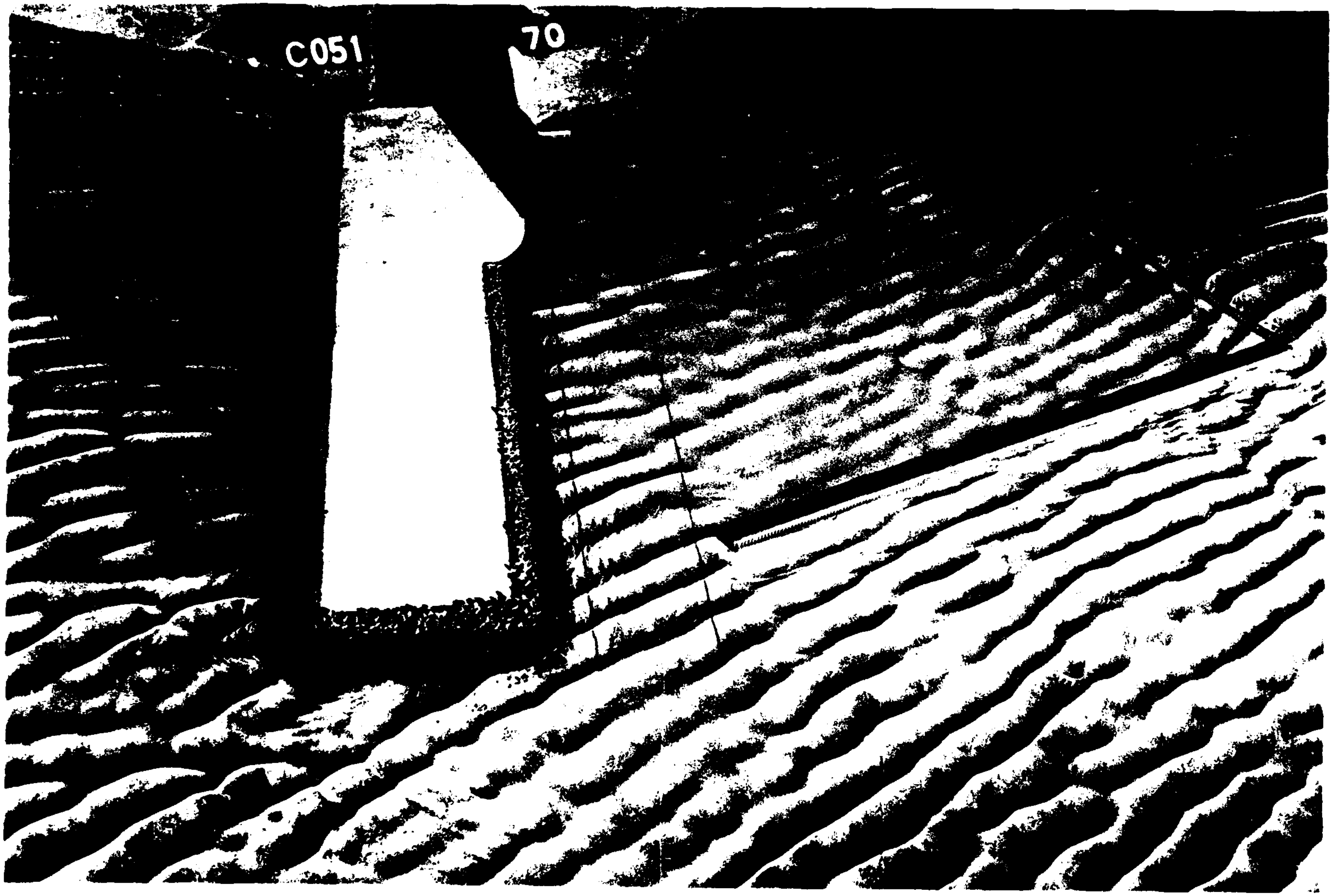

Photo 61. Typical wave patterns for Plan 11; 3.5-sec, 2.4-ft waves from 125 deg

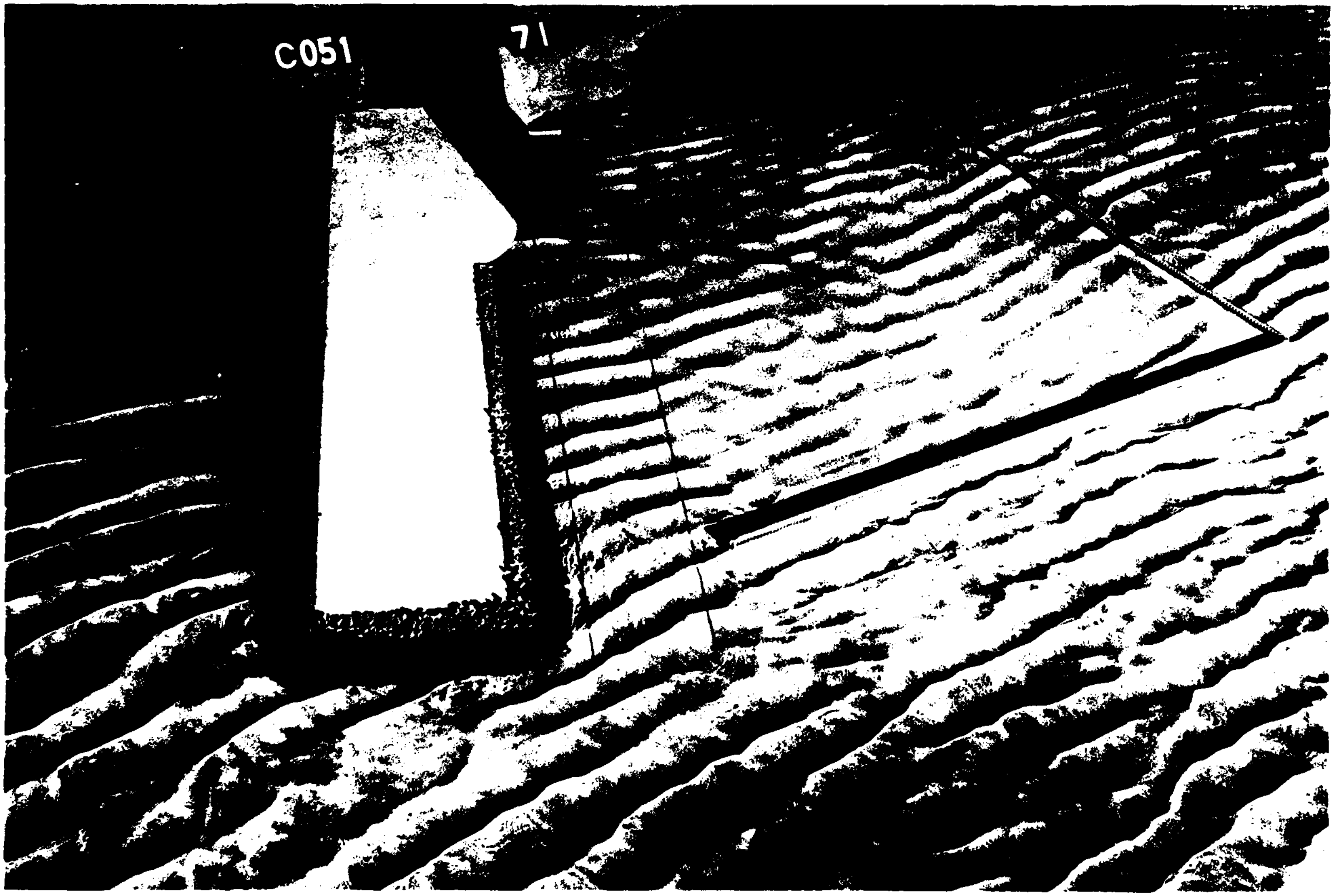

Photo 62. Typical wave patterns for Plan 11; 3.6-sec, 3.8-ft waves from 125 deg

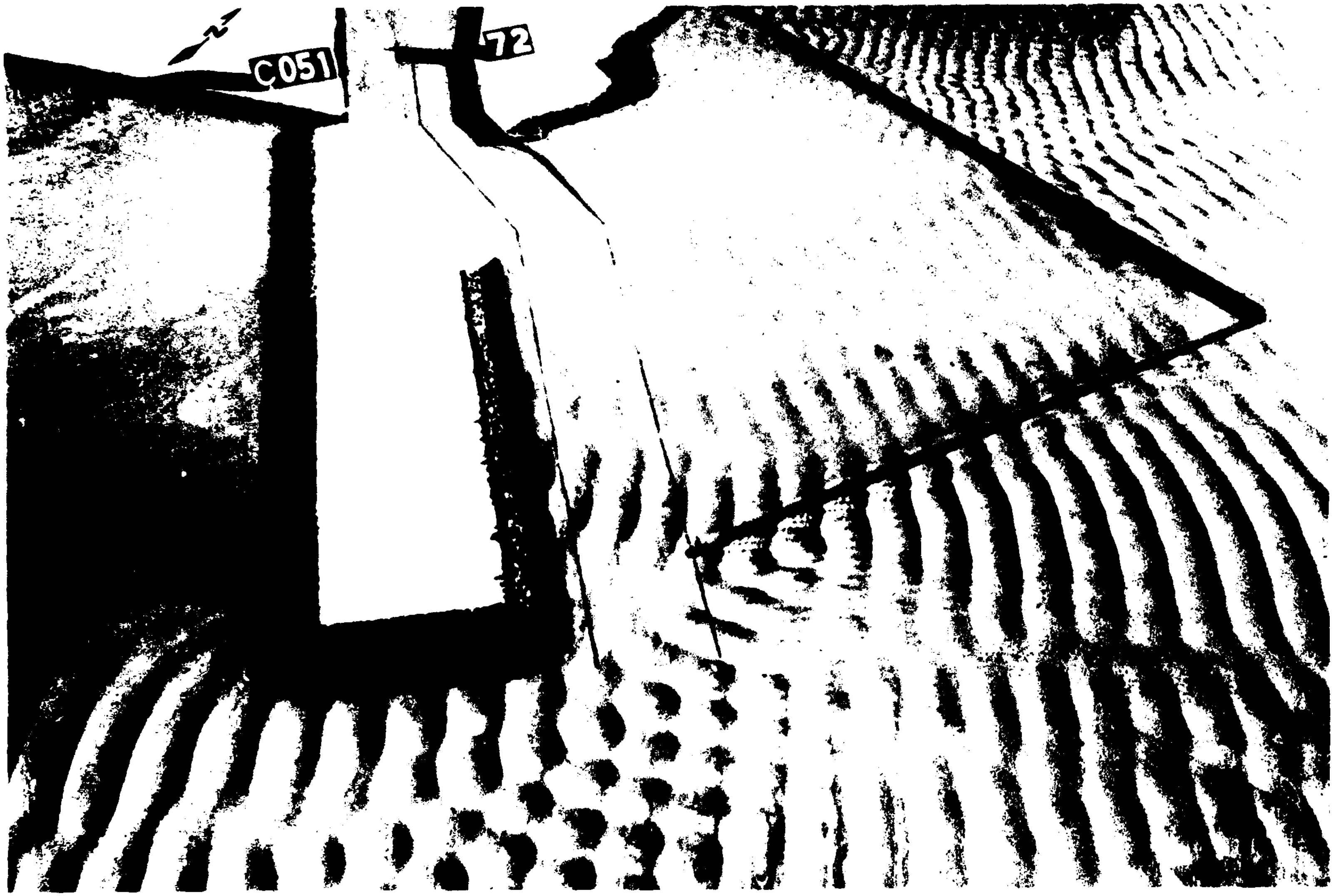

Photo 63. Typical wave patterns for Plan 11; 4.3-sec, 4-ft waves from 70 deg

Photo 64. Typical wave patterns for Plan 11; 6-sec, 9-ft waves from 70 deg

AD-A148 185 IMPACT OF I-664 BRIDGE/TUNNEL PROJECT ON WAVE CONDITIONS AT NEWPORT NEWS..(U) COASTAL ENGINEERING RESEARCH CENTER VICKSBURG MS R R BOTTIN OCT 84 2/2

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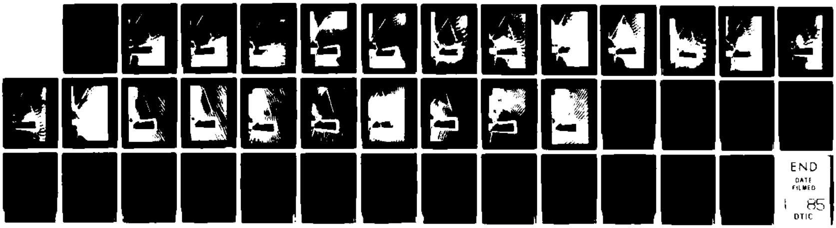
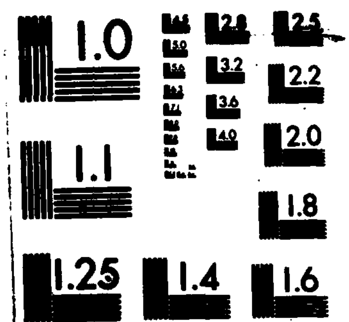




MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

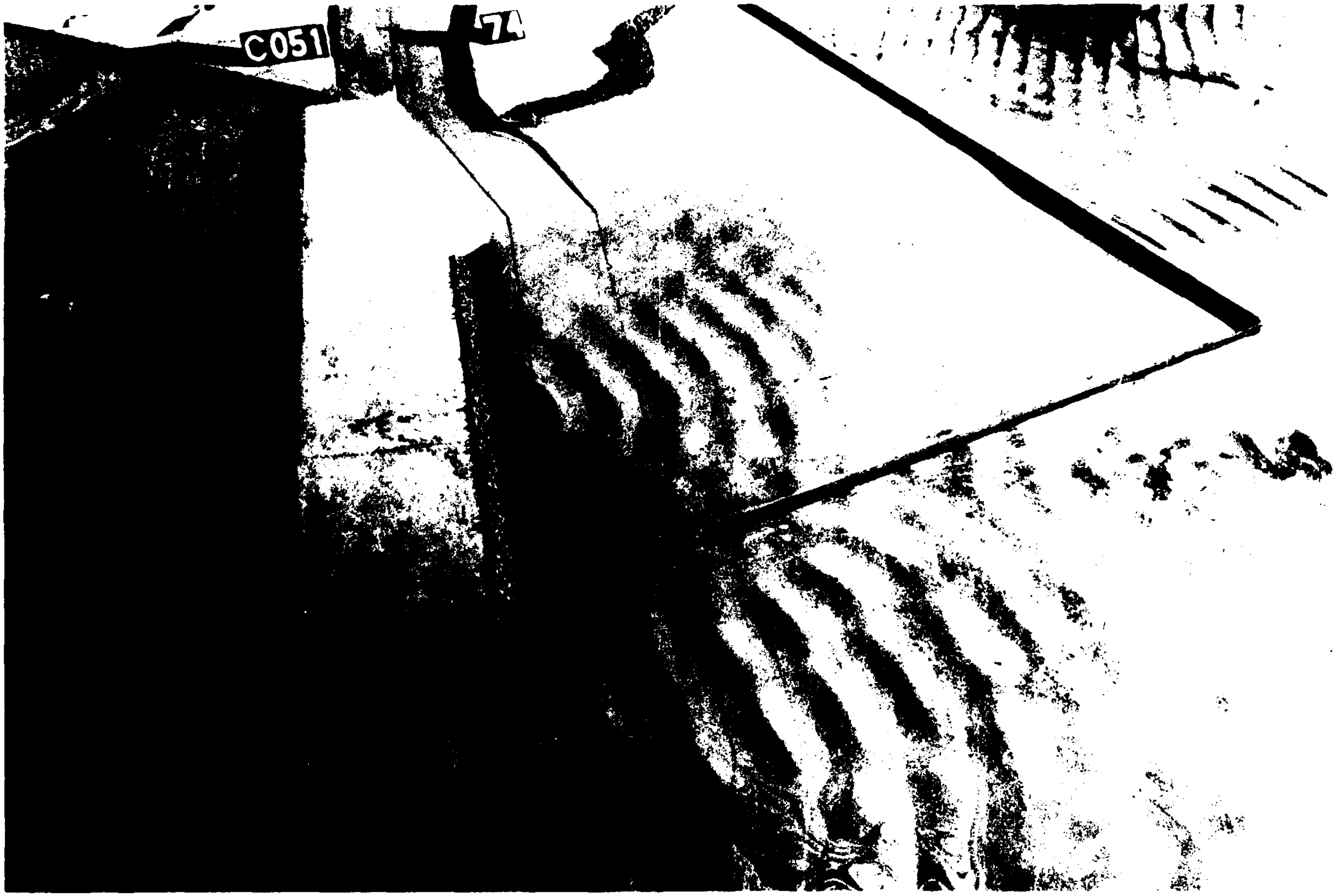


Photo 65. Typical wave patterns for Plan 12; 4.3-sec, 4-ft waves from 70 deg

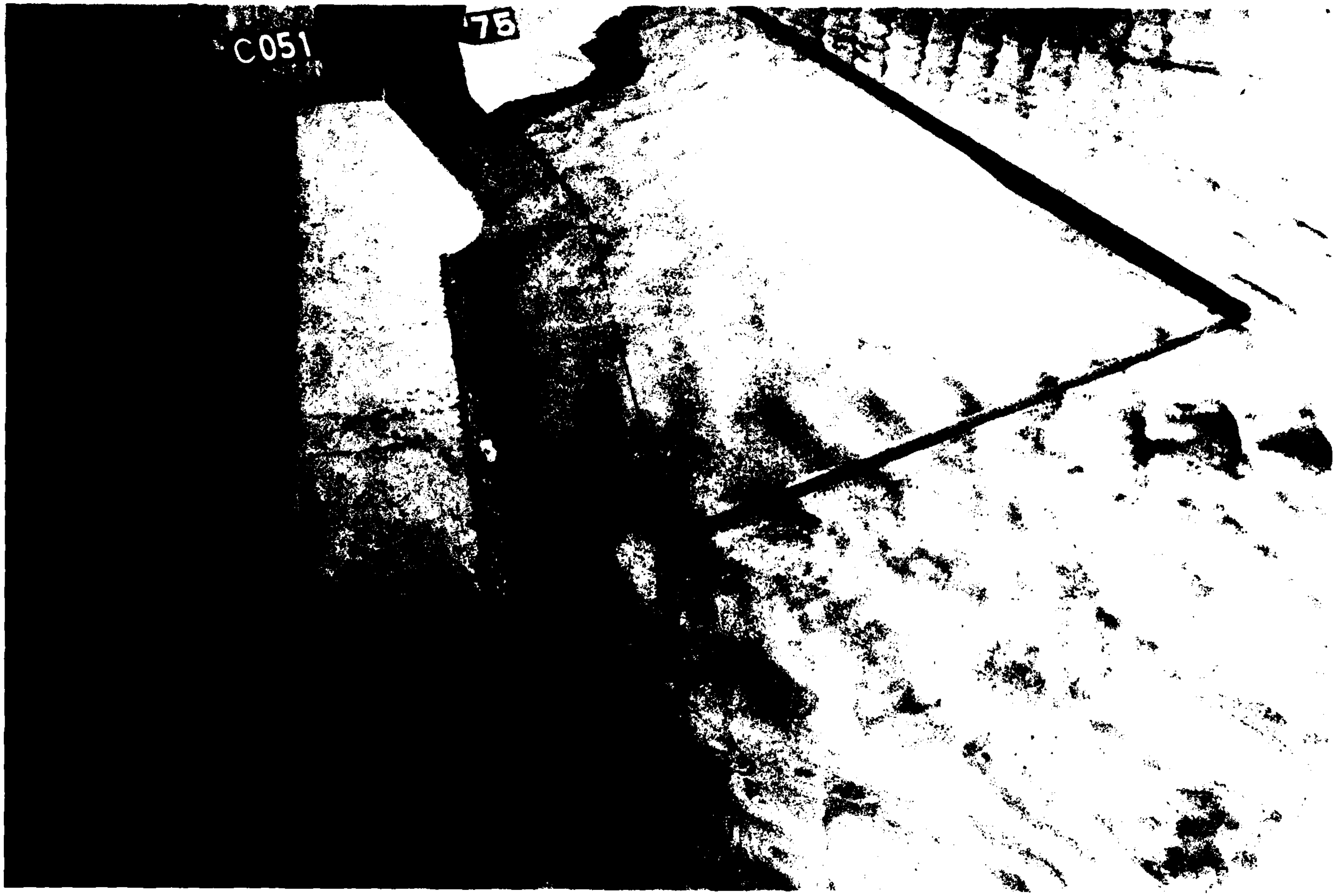


Photo 66. Typical wave patterns for Plan 12; 6-sec, 9-ft waves from 70 deg

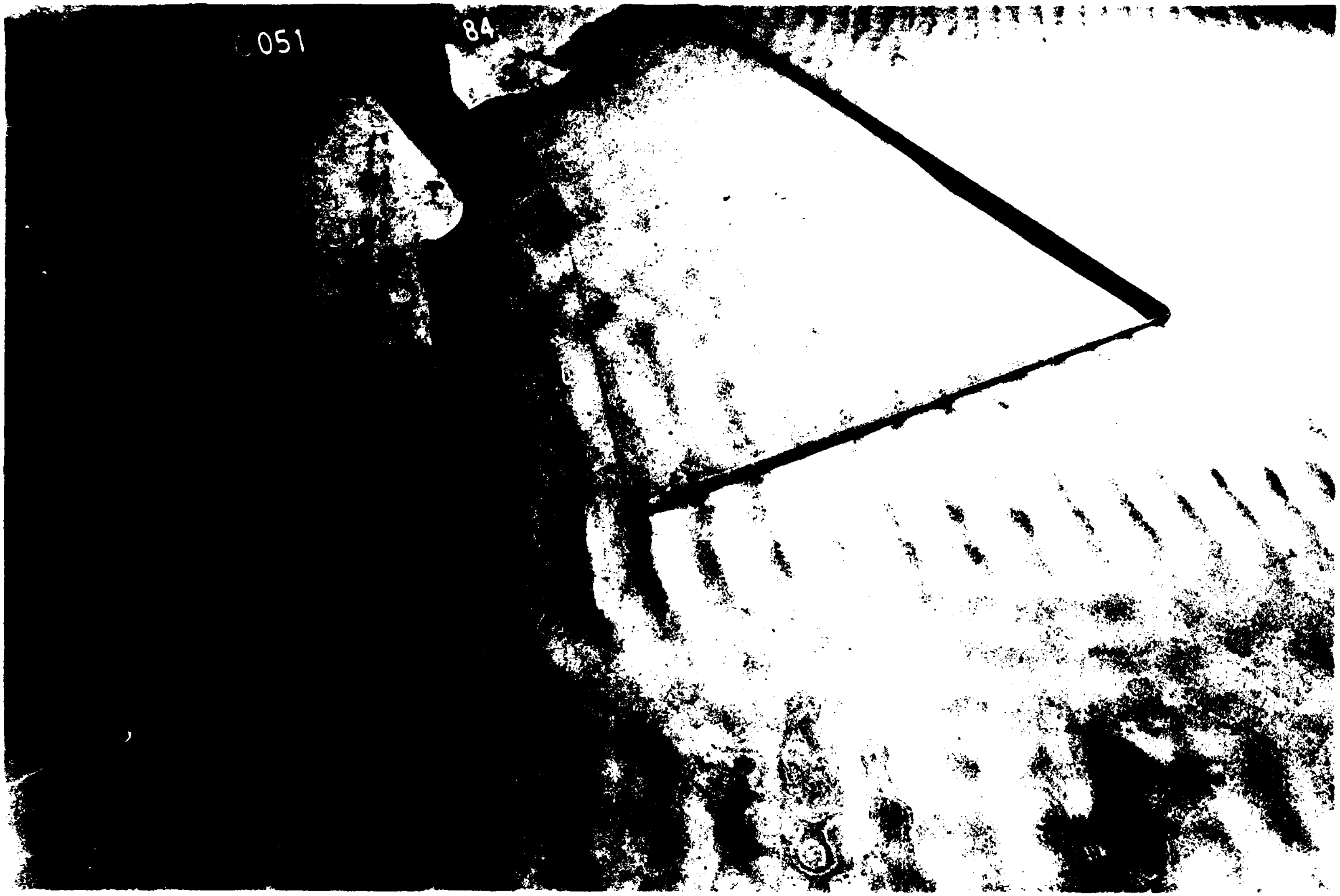


Photo 67. Typical wave patterns for Plan 13; 4.3-sec, 4-ft waves from 70 deg

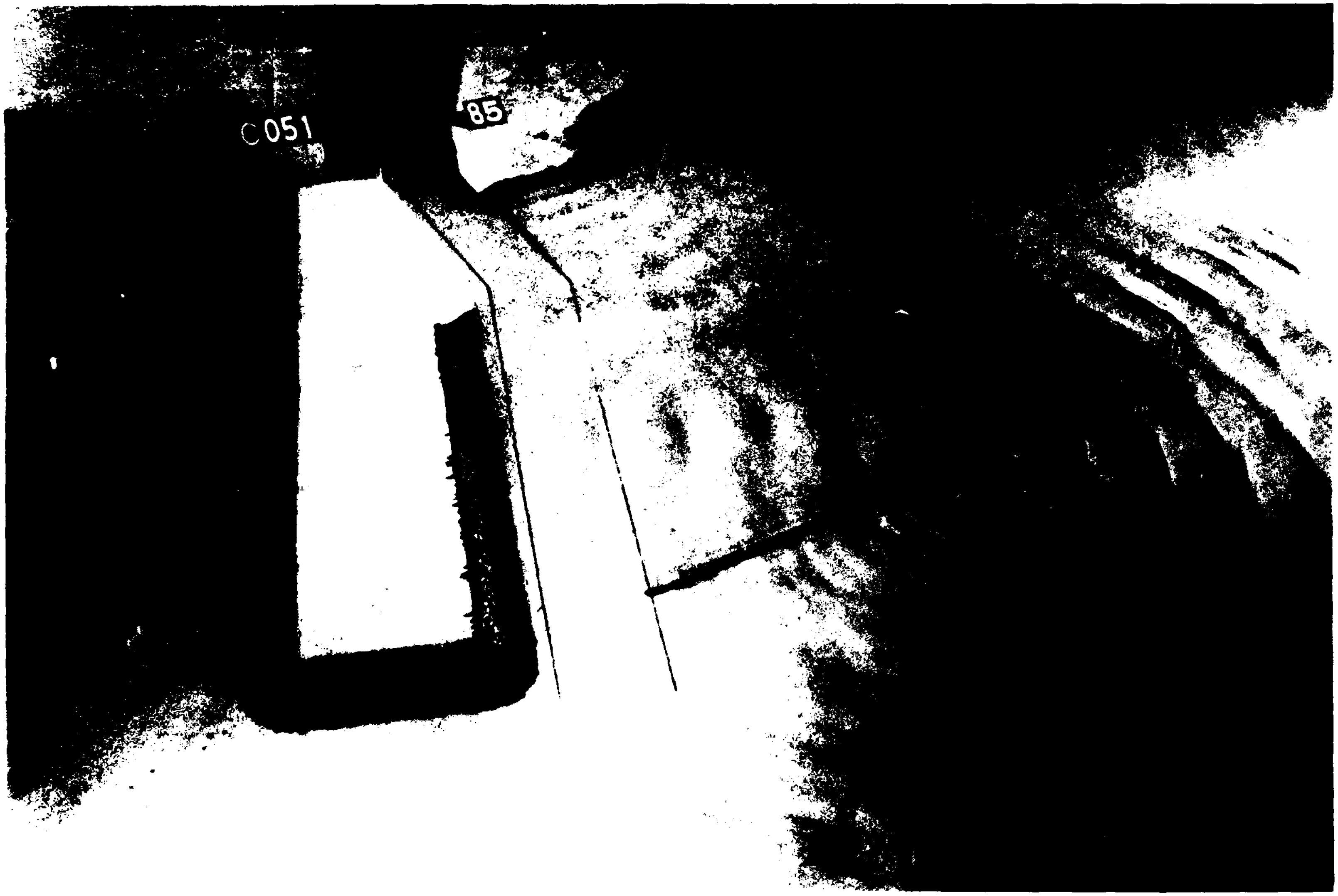


Photo 68. Typical wave patterns for Plan 13; 6-sec, 9-ft waves from 70 deg

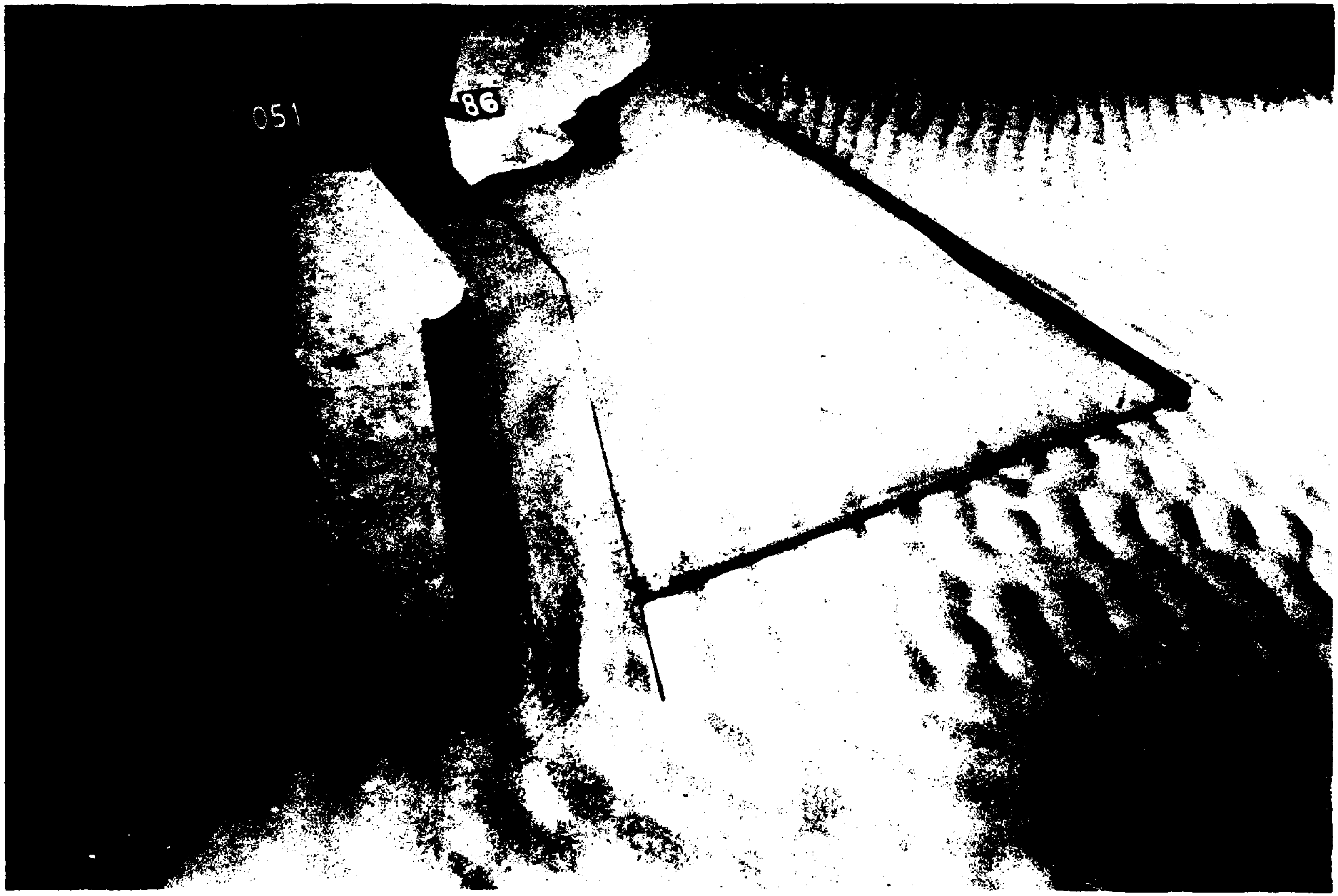


Photo 69. Typical wave patterns for Plan 14; 4.3-sec, 4-ft waves from 70 deg

Photo 70. Typical wave patterns for Plan 14; 6-sec, 9-ft waves from 70 deg

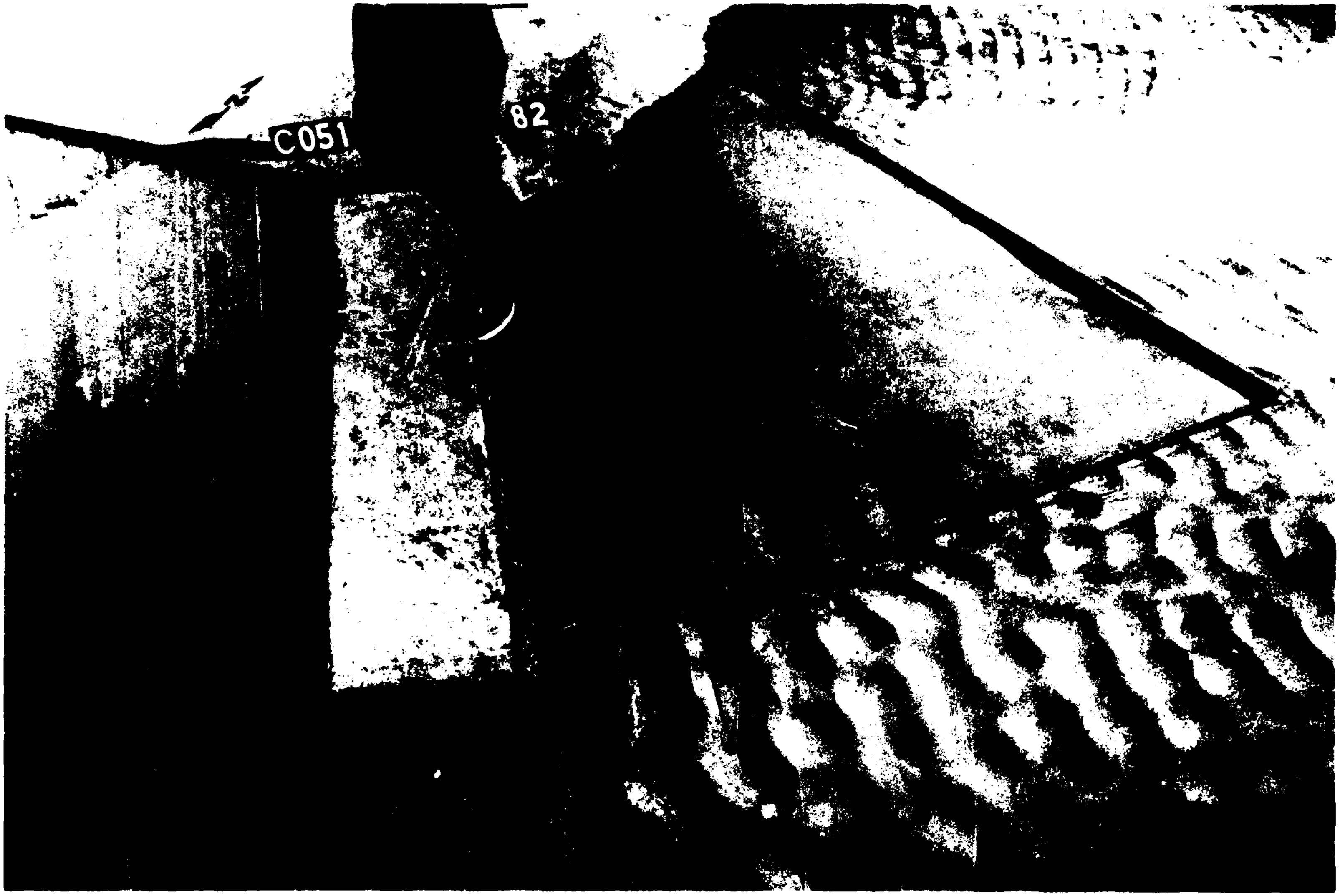


Photo 71. Typical wave patterns for Plan 15; 4.3-sec, 4-ft waves from 70 deg

Photo 72. Typical wave patterns for Plan 15; 6-sec, 9-ft waves from 70 deg

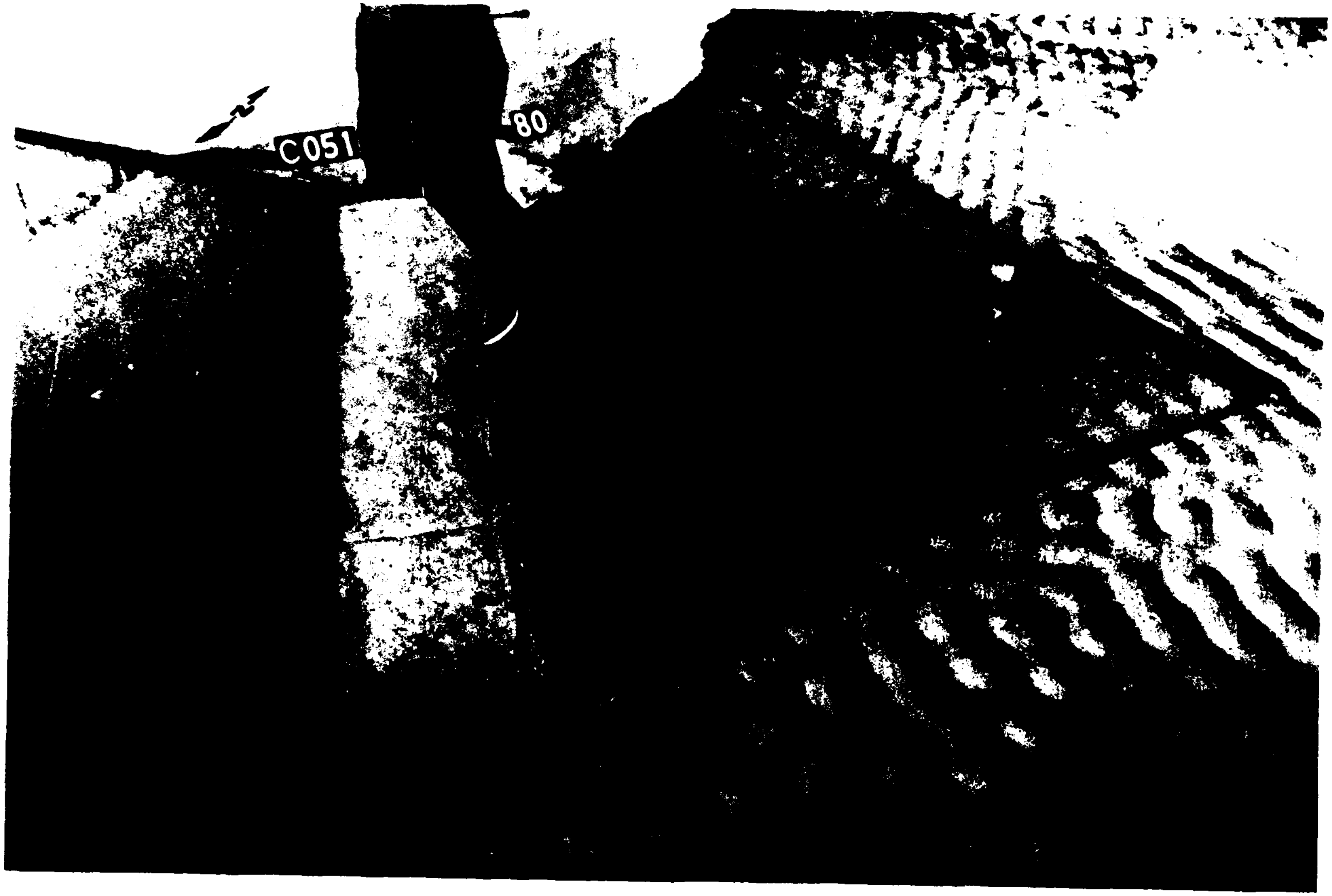


Photo 73. Typical wave patterns for Plan 16; 4.3-sec, 4-ft waves from 70 deg

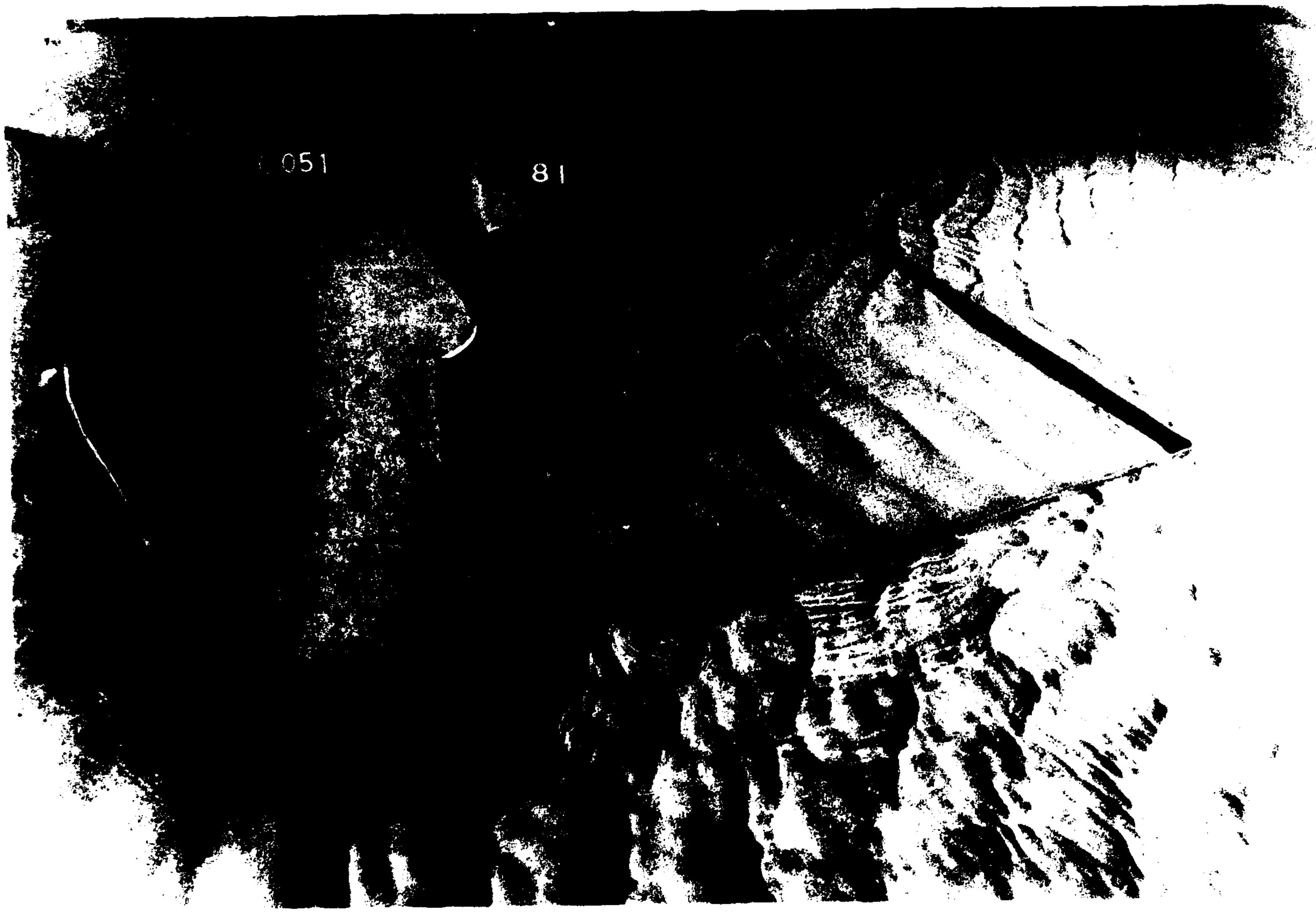


Photo 74. Typical wave patterns for Plan 16; 6-sec, 9-ft waves from 70 deg

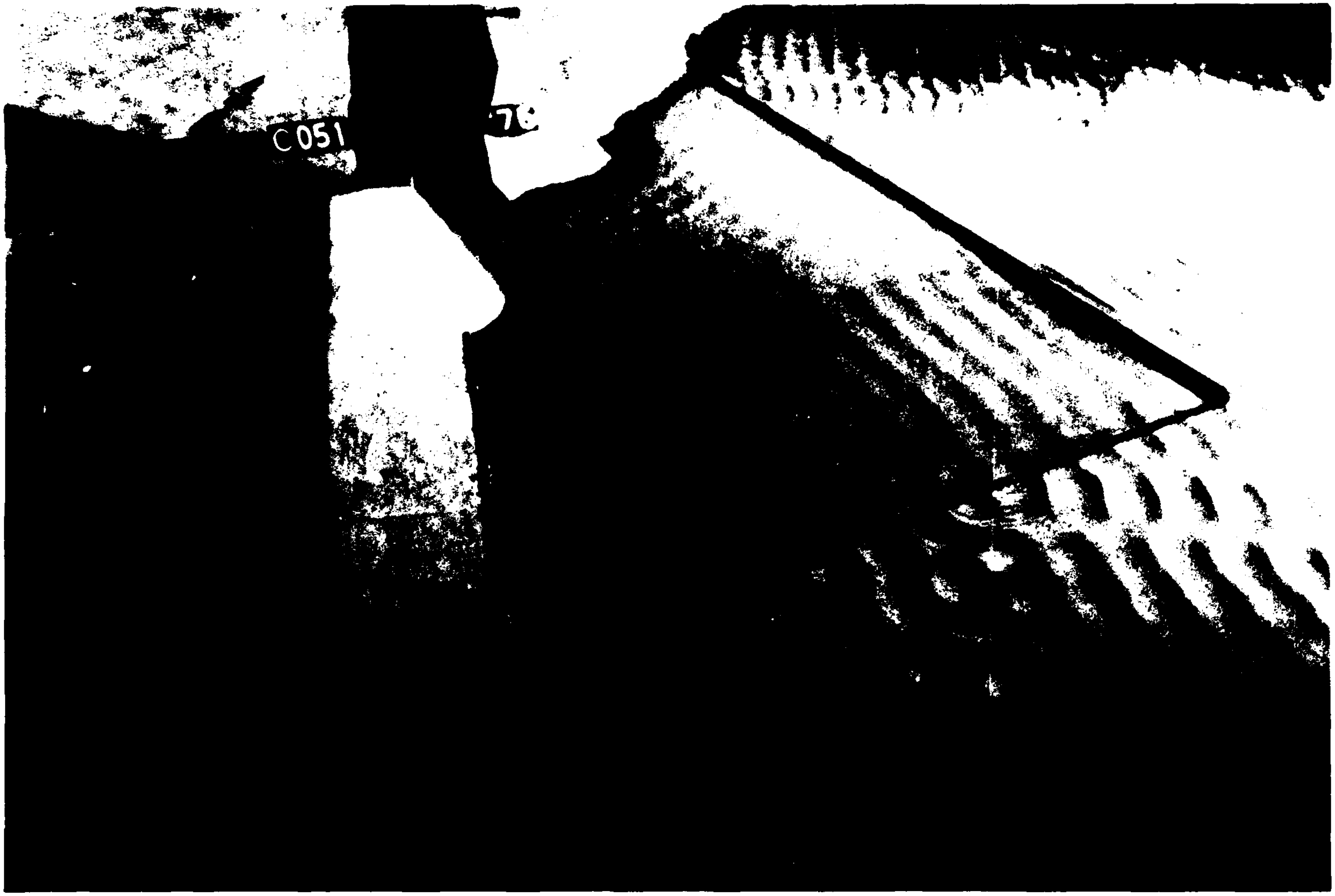


Photo 75. Typical wave patterns for Plan 17; 4.3-sec, 4-ft waves from 70 deg

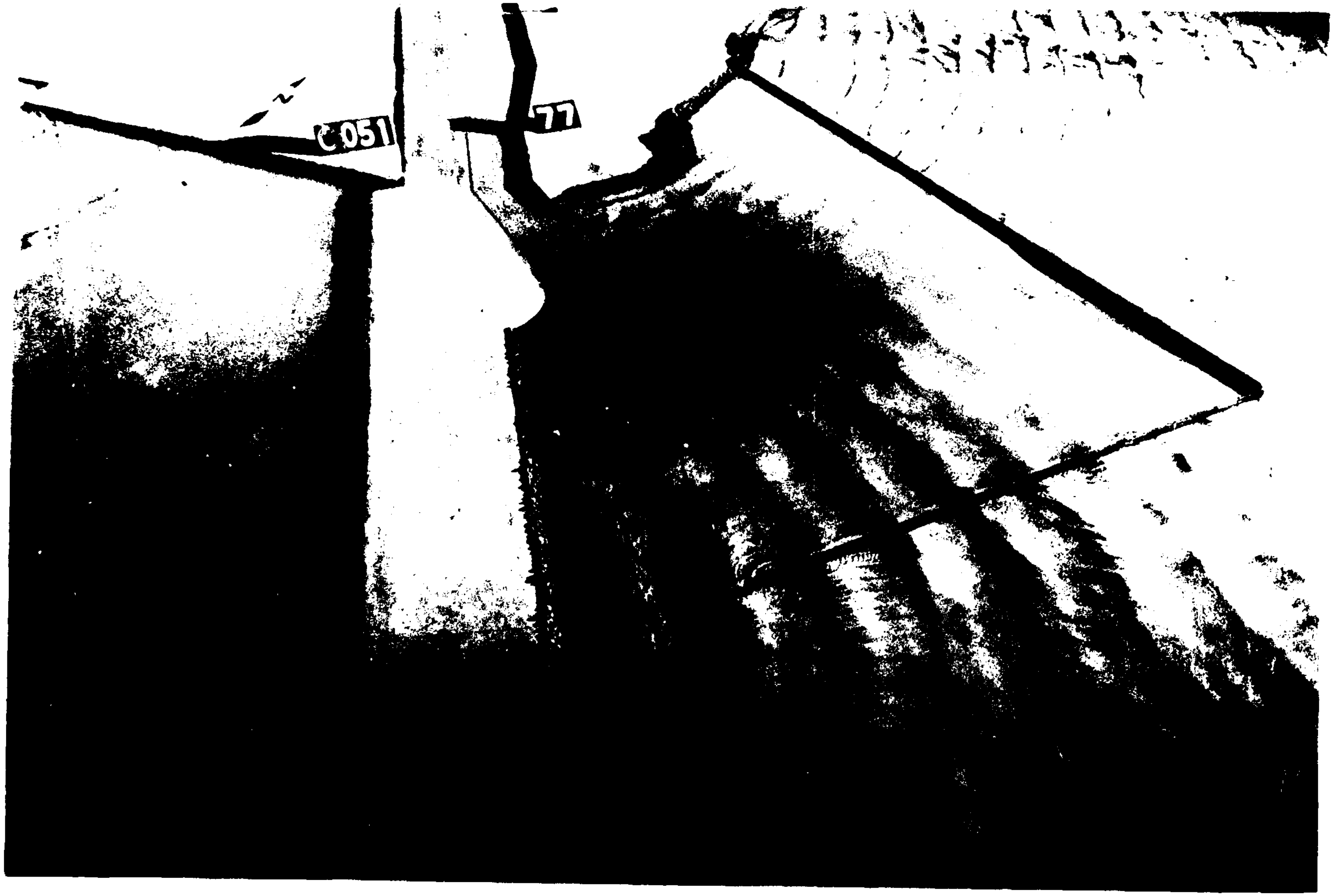


Photo 76. Typical wave patterns for Plan 17; 6-sec, 9-ft waves from 70 deg

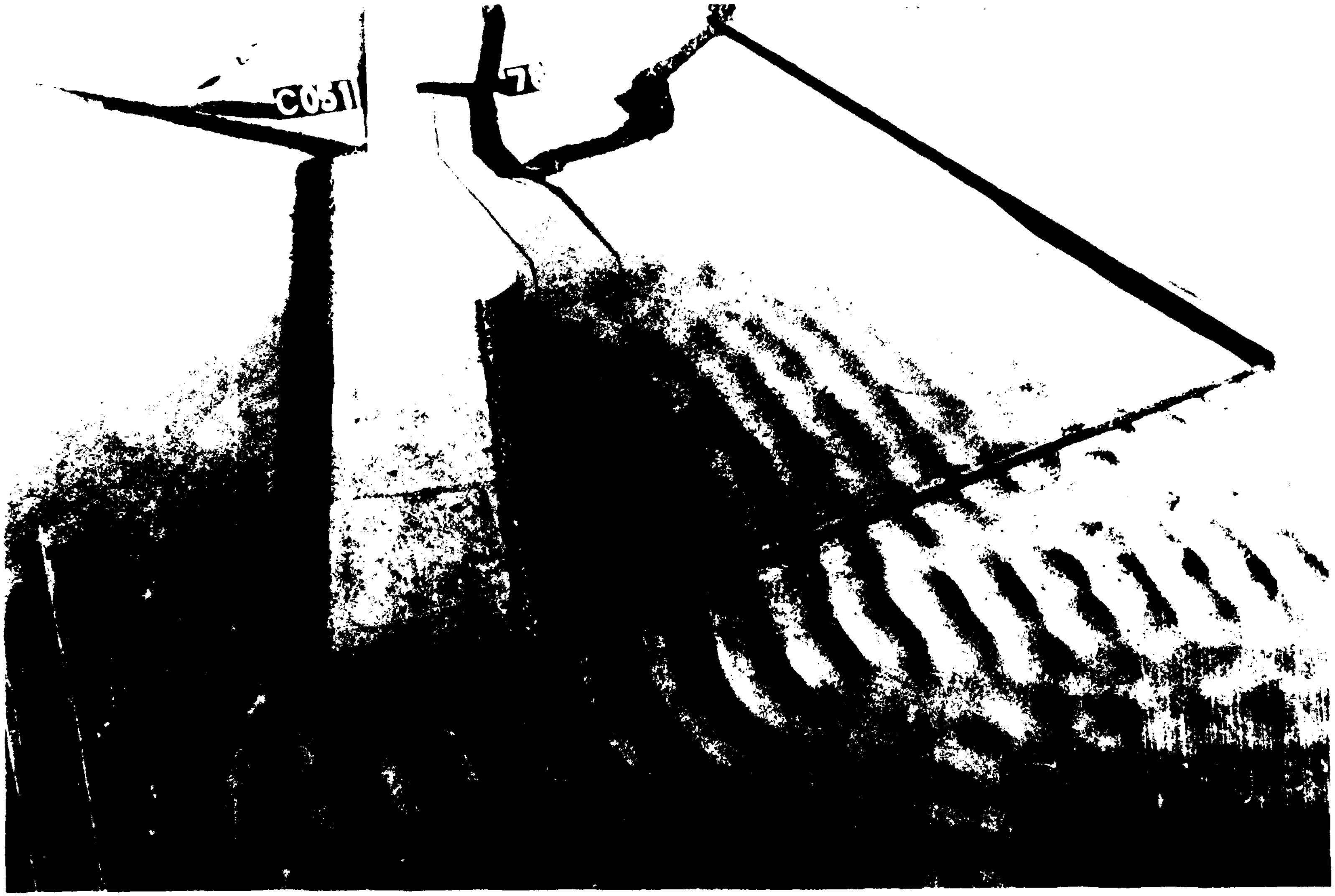


Photo 77. Typical wave patterns for Plan 18; 4.3-sec, 4-ft waves from 70 deg

Photo 78. Typical wave patterns for Plan 18; 6-sec, 9-ft waves from 70 deg

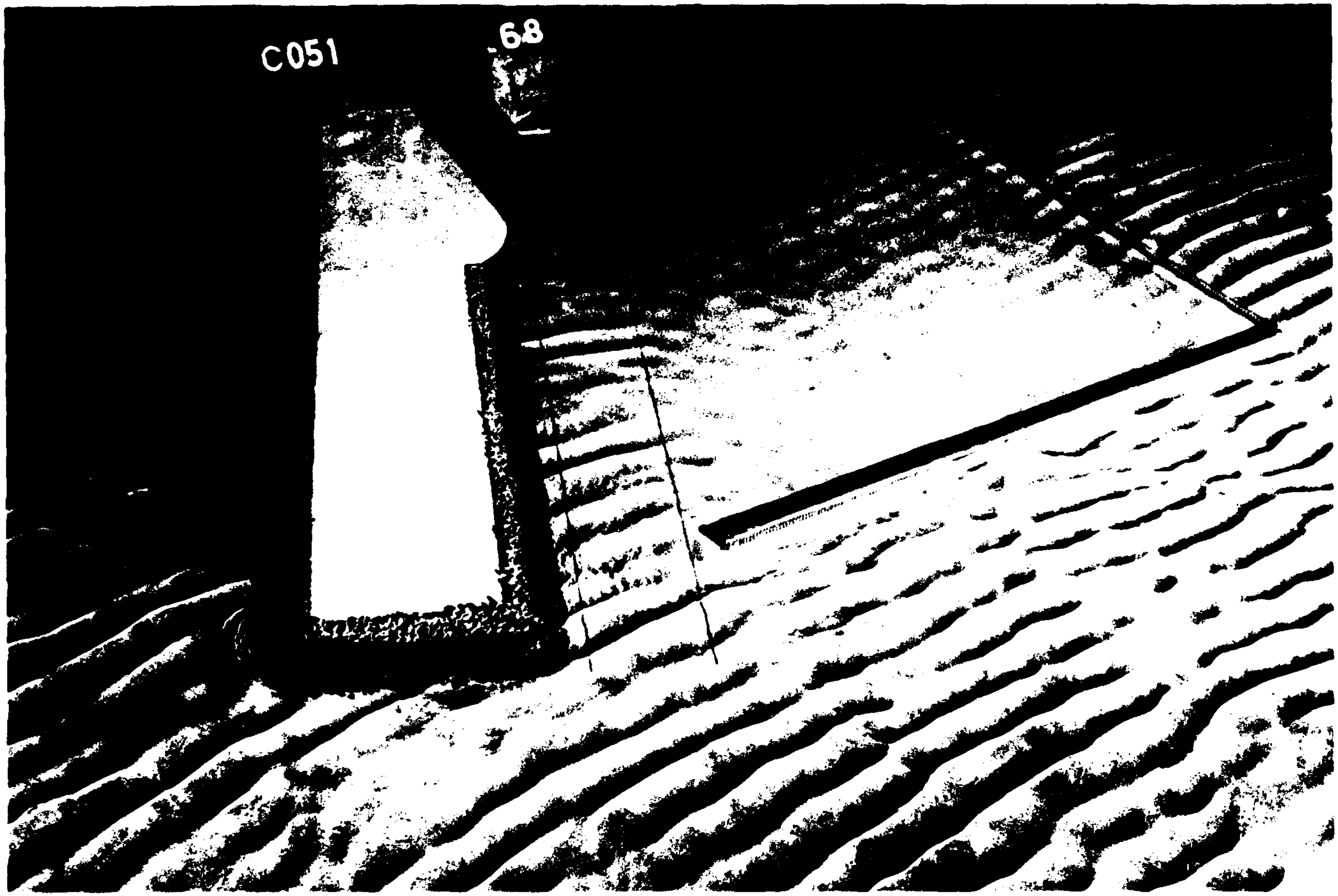


Photo 79. Typical wave patterns for Plan 12; 3.5-sec, 2.4-ft waves from 125 deg

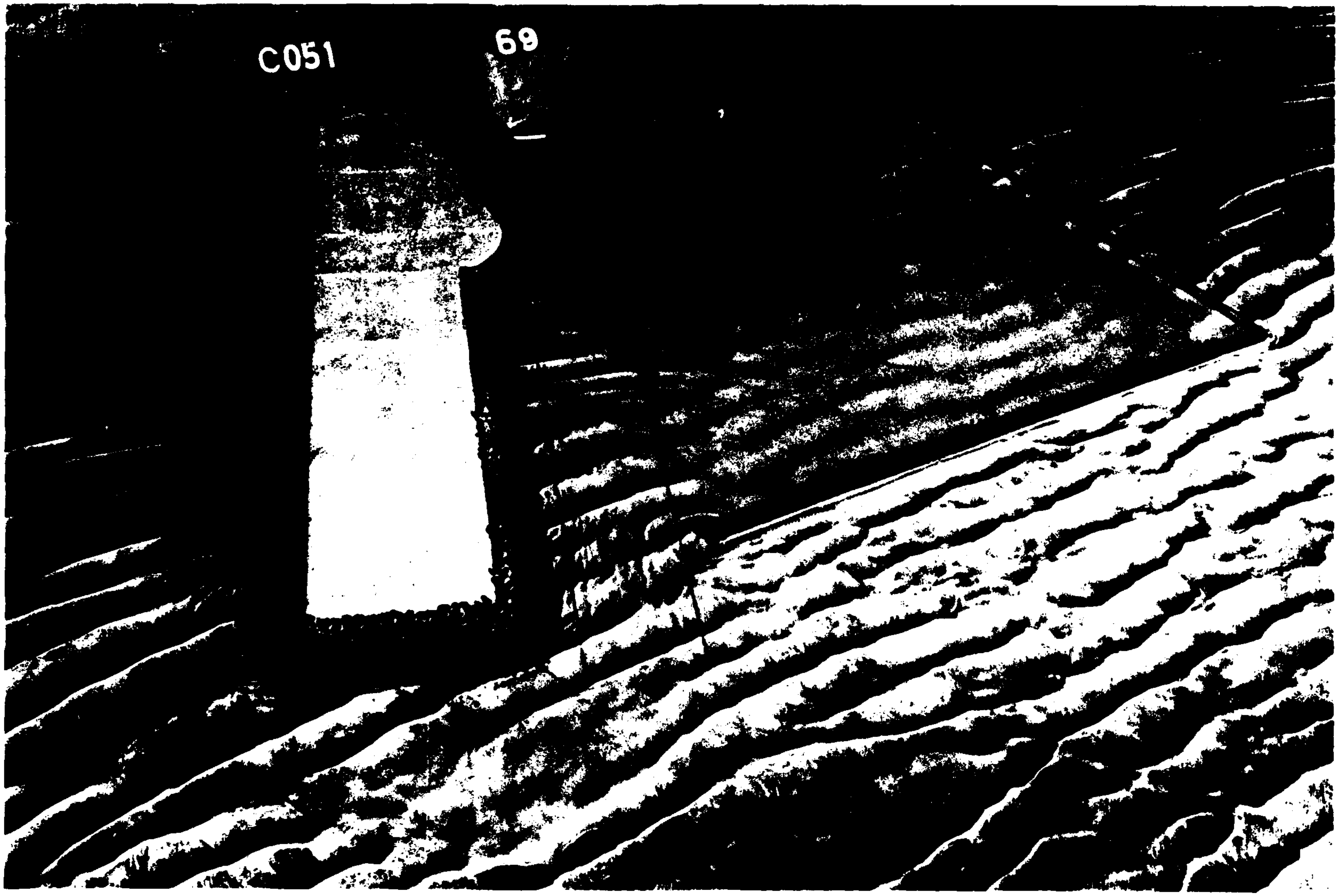


Photo 80. Typical wave patterns for Plan 12; 3.6-sec, 3.8-ft waves from 125 deg

Photo 81. Typical wave patterns for Plan 12; 3.5-sec, 1.7-ft waves from 140 deg

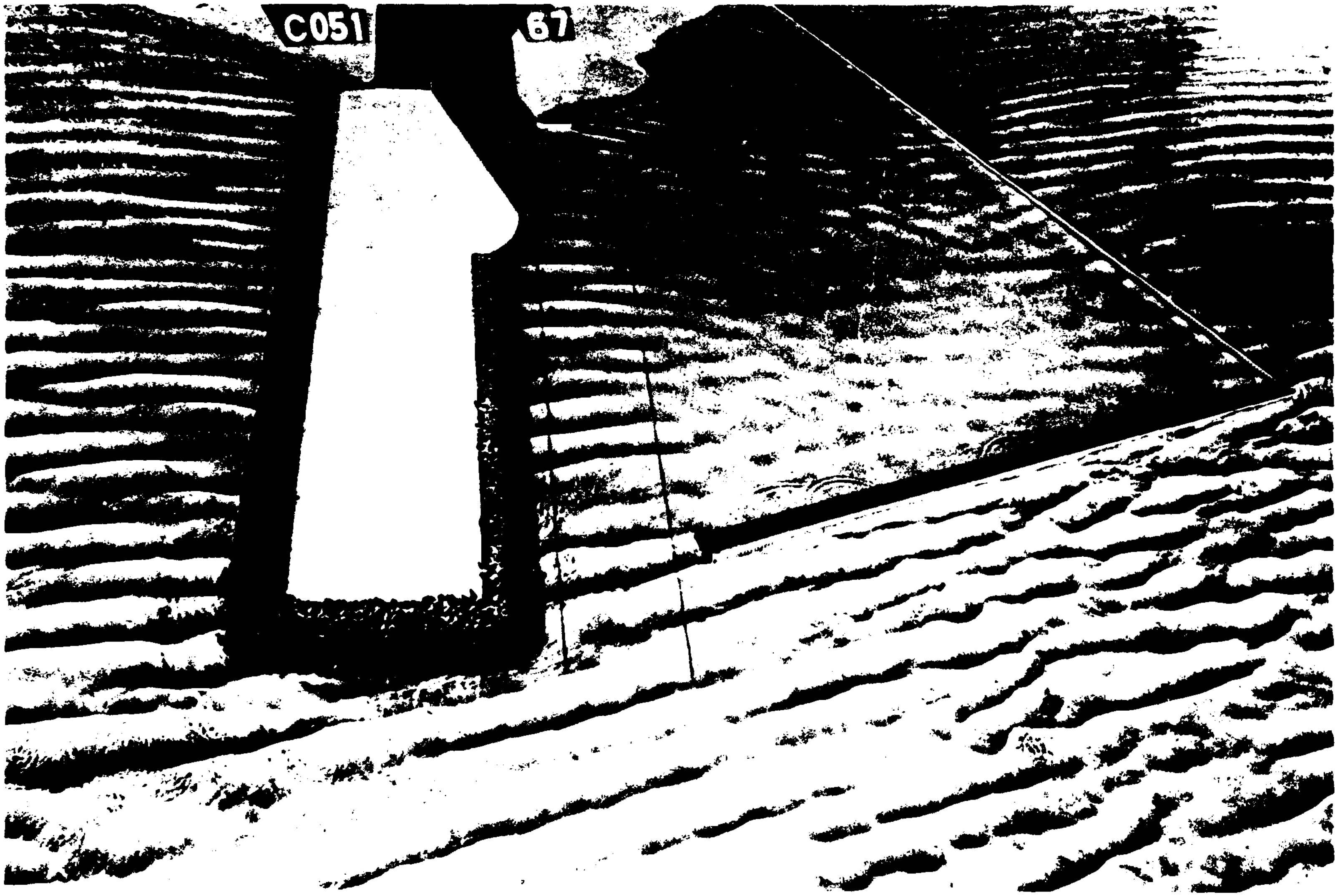


Photo 82. Typical wave patterns for Plan 12; 3.5-sec, 3.7-ft waves from 140 deg

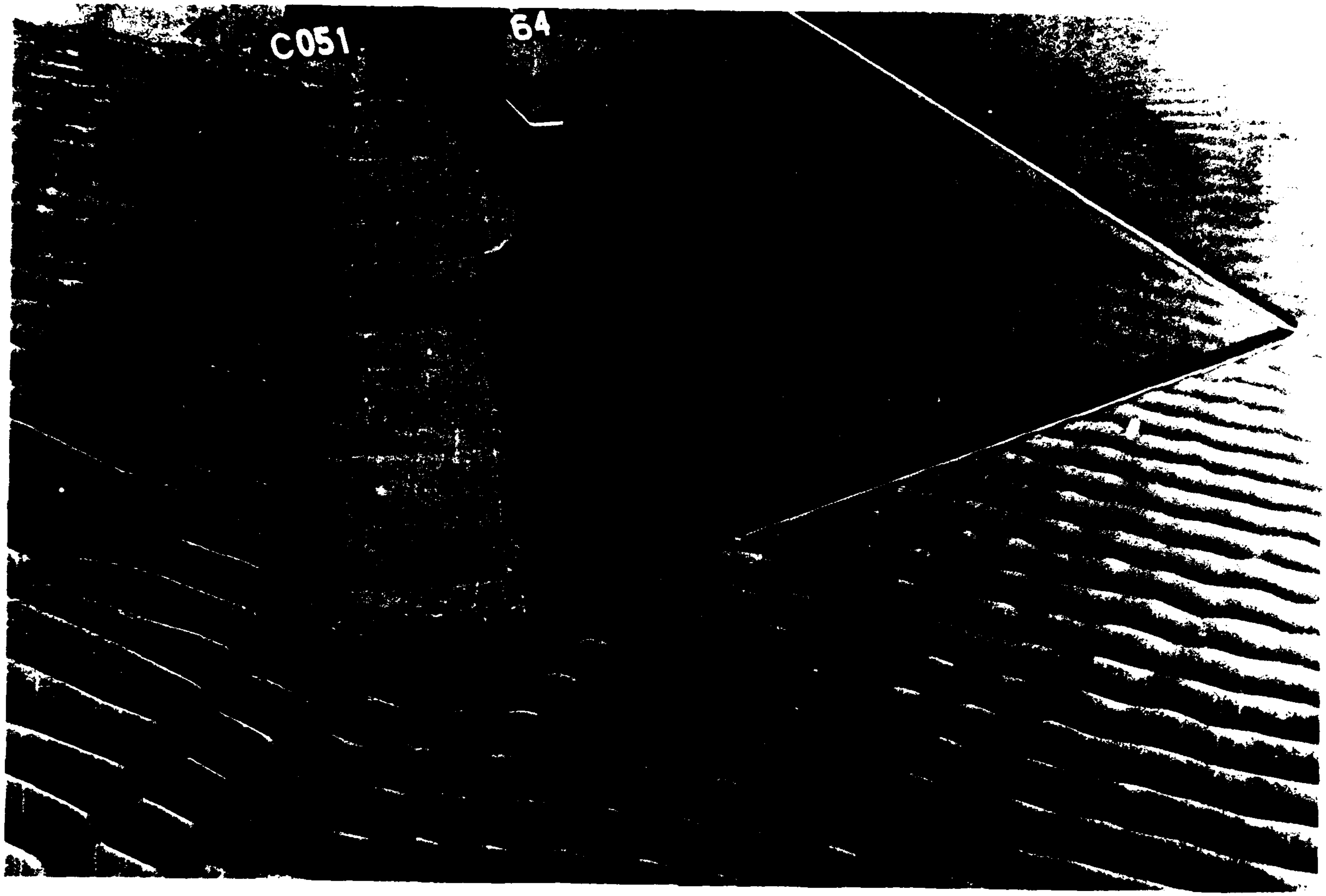


Photo 83. Typical wave patterns for Plan 12; 3.5-sec, 2.3-ft waves from 180 deg

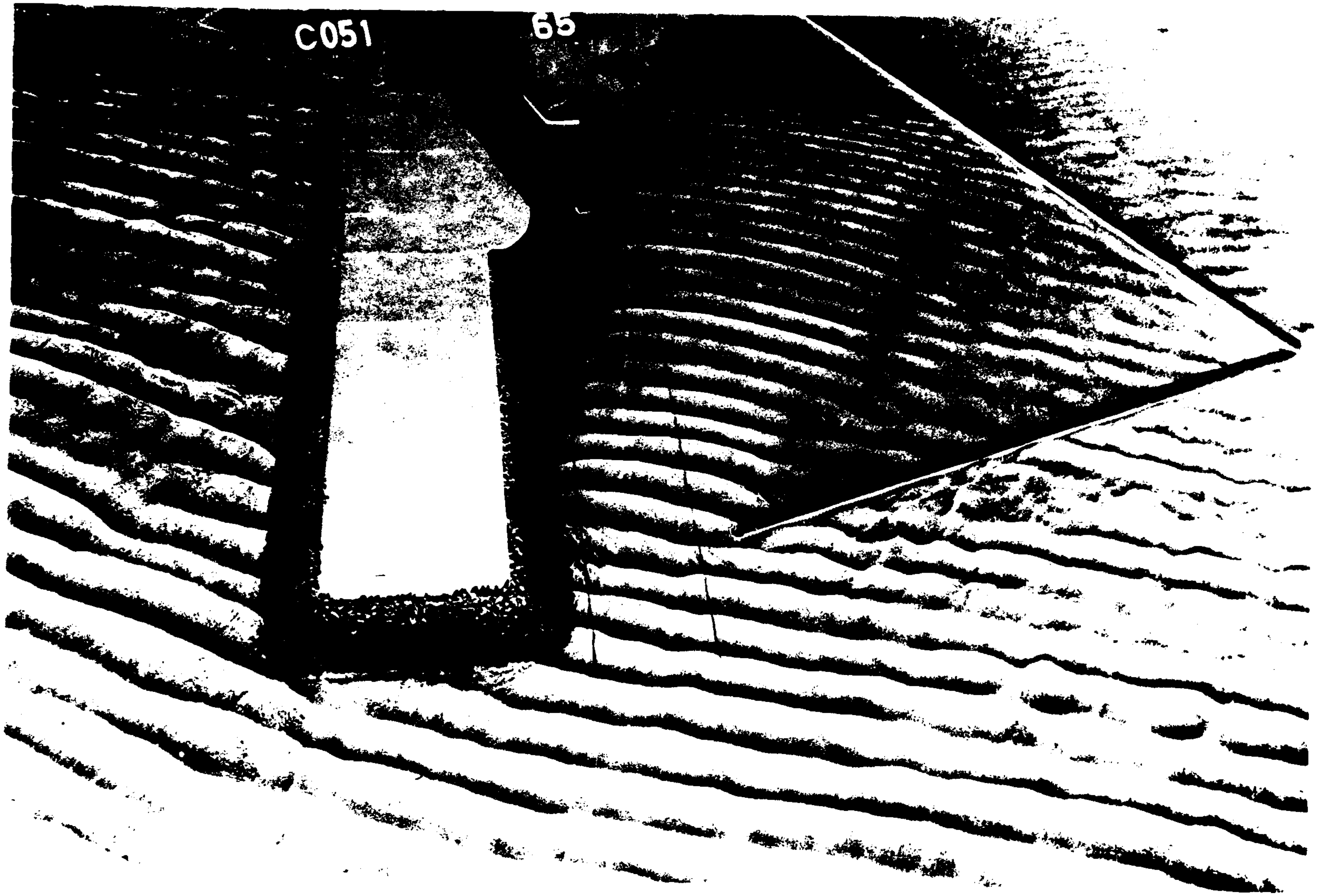


Photo 84. Typical wave patterns for Plan 12; 3.6-sec, 4.1-ft waves from 180 deg

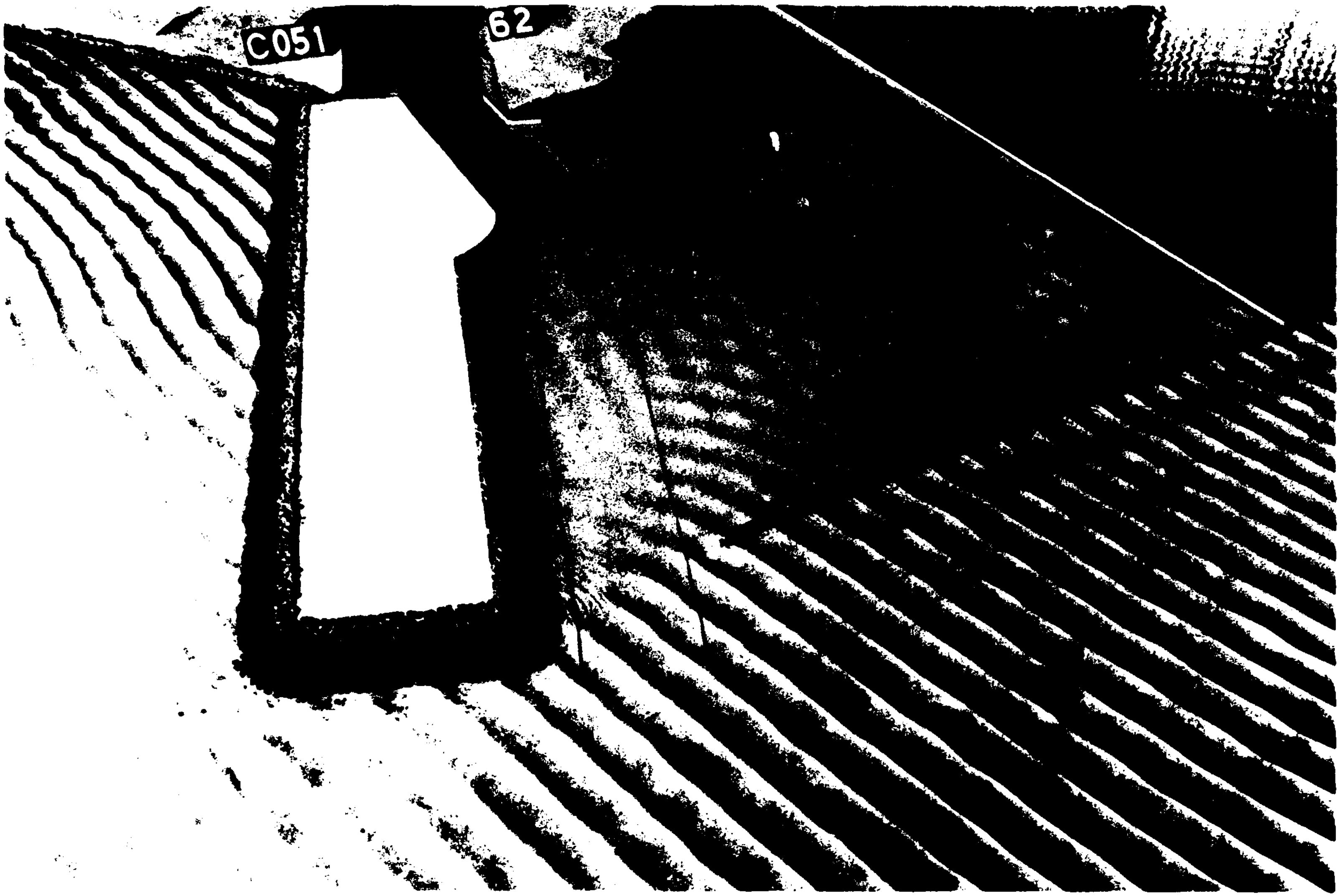


Photo 85. Typical wave patterns for Plan 12; 3.5-sec, 2.5-ft waves from 215 deg

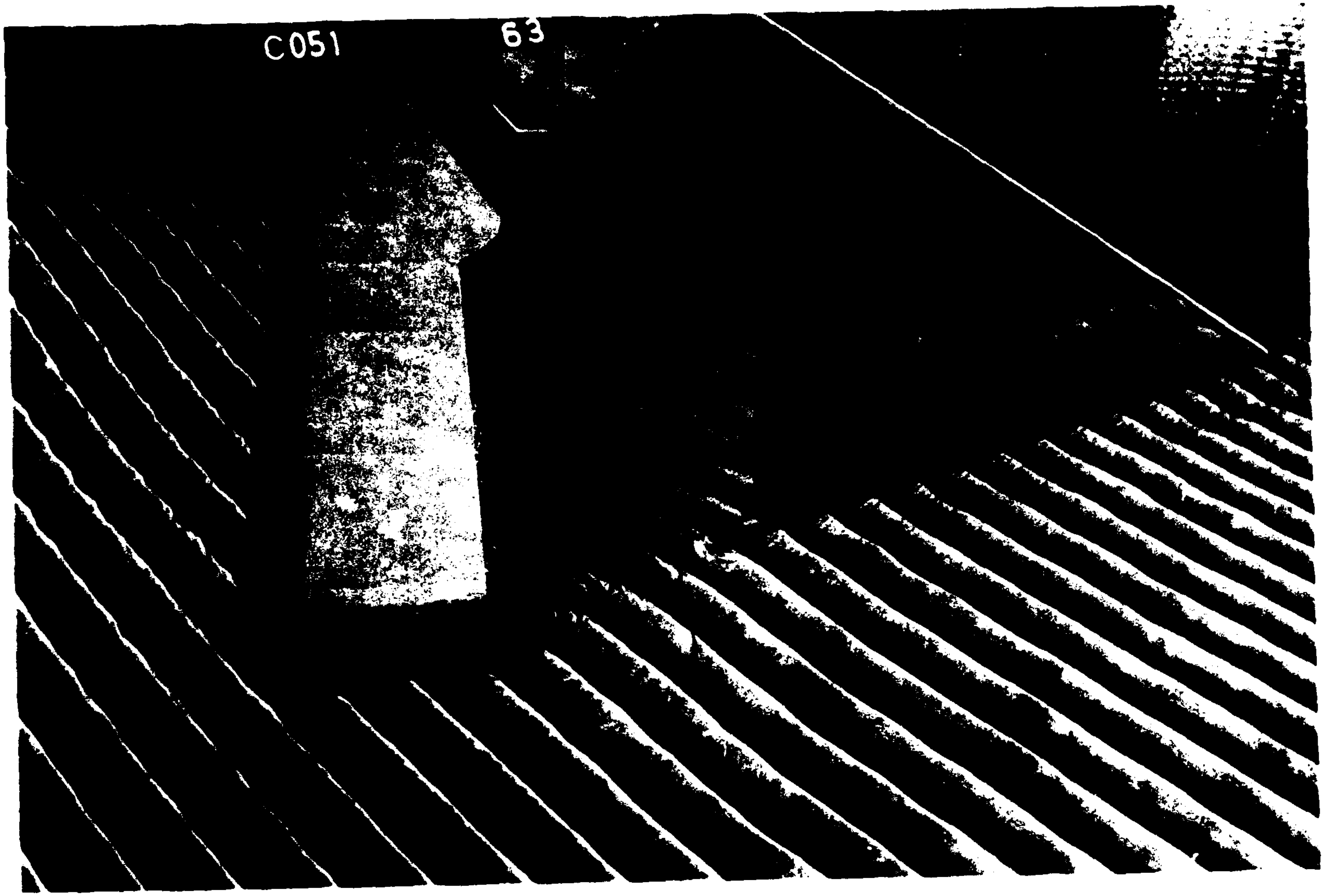
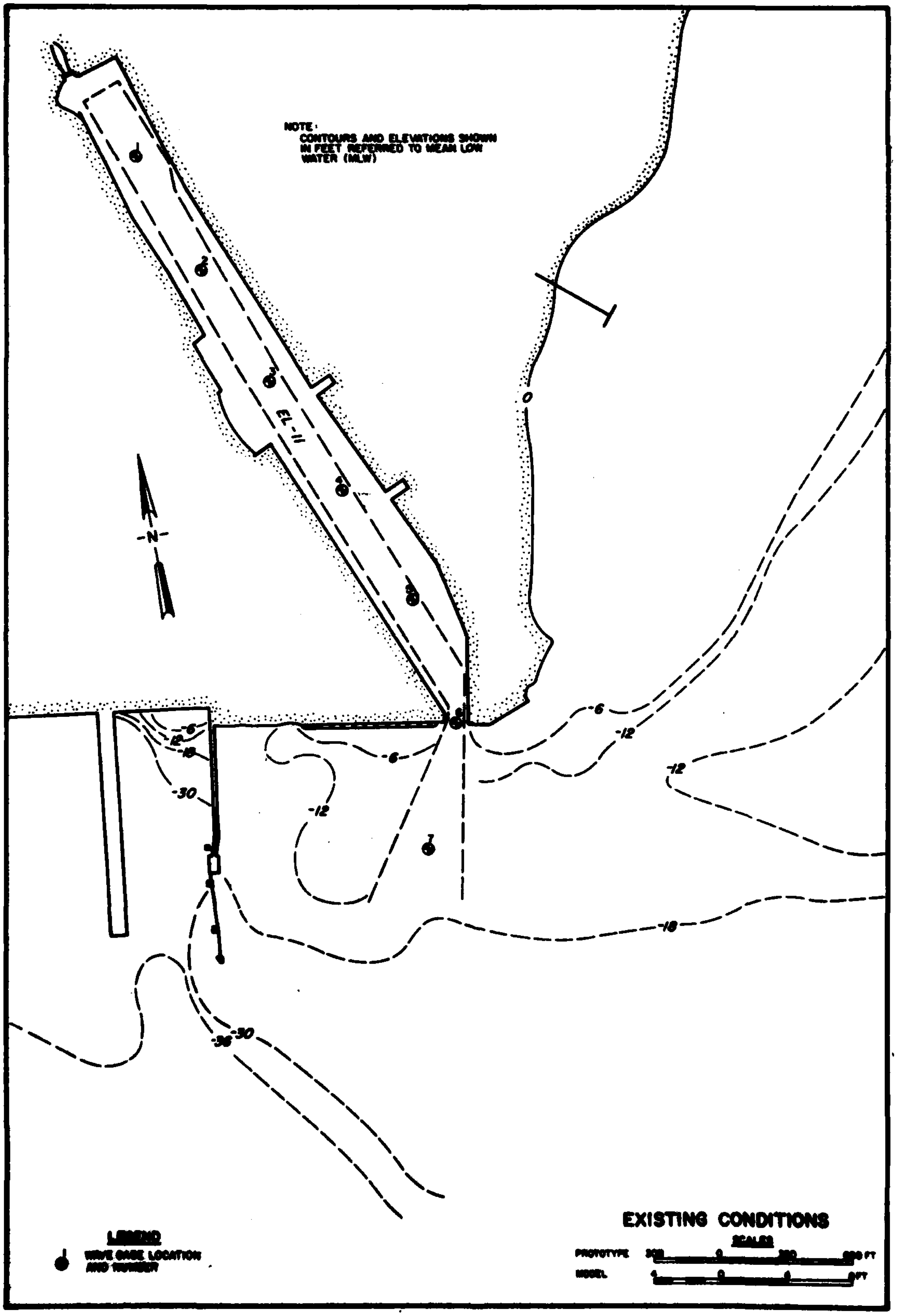


Photo 86. Typical wave patterns for Plan 12; 3.9-sec, 4.5-ft waves from 215 deg



NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
EL-11
N
0
-6
-12
-18
-30
-36
LEGEND
WAVE GAGE LOCATION AND NUMBER
EXISTING CONDITIONS
SCALES
PROTOTYPE
MODEL

PLATE 1

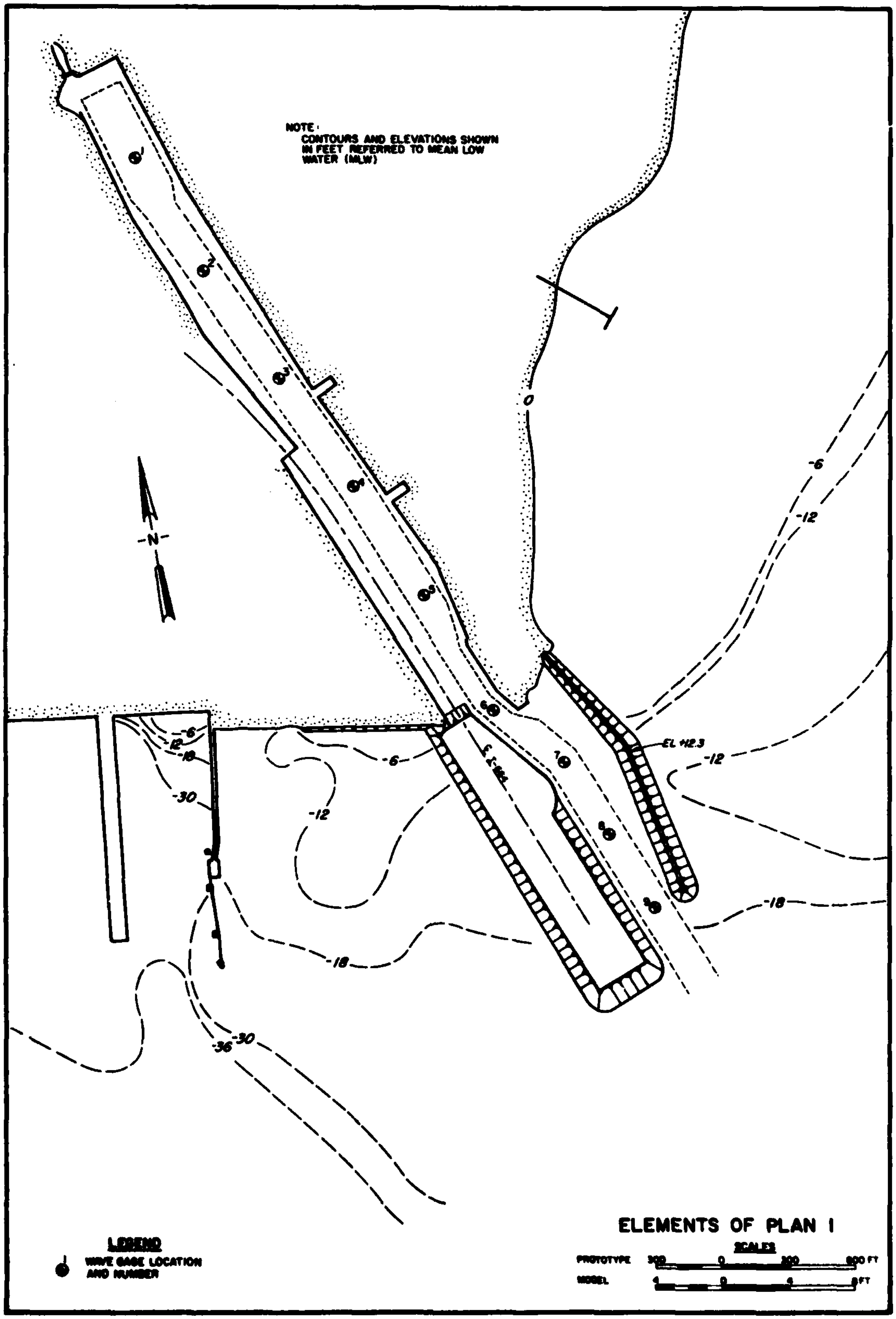
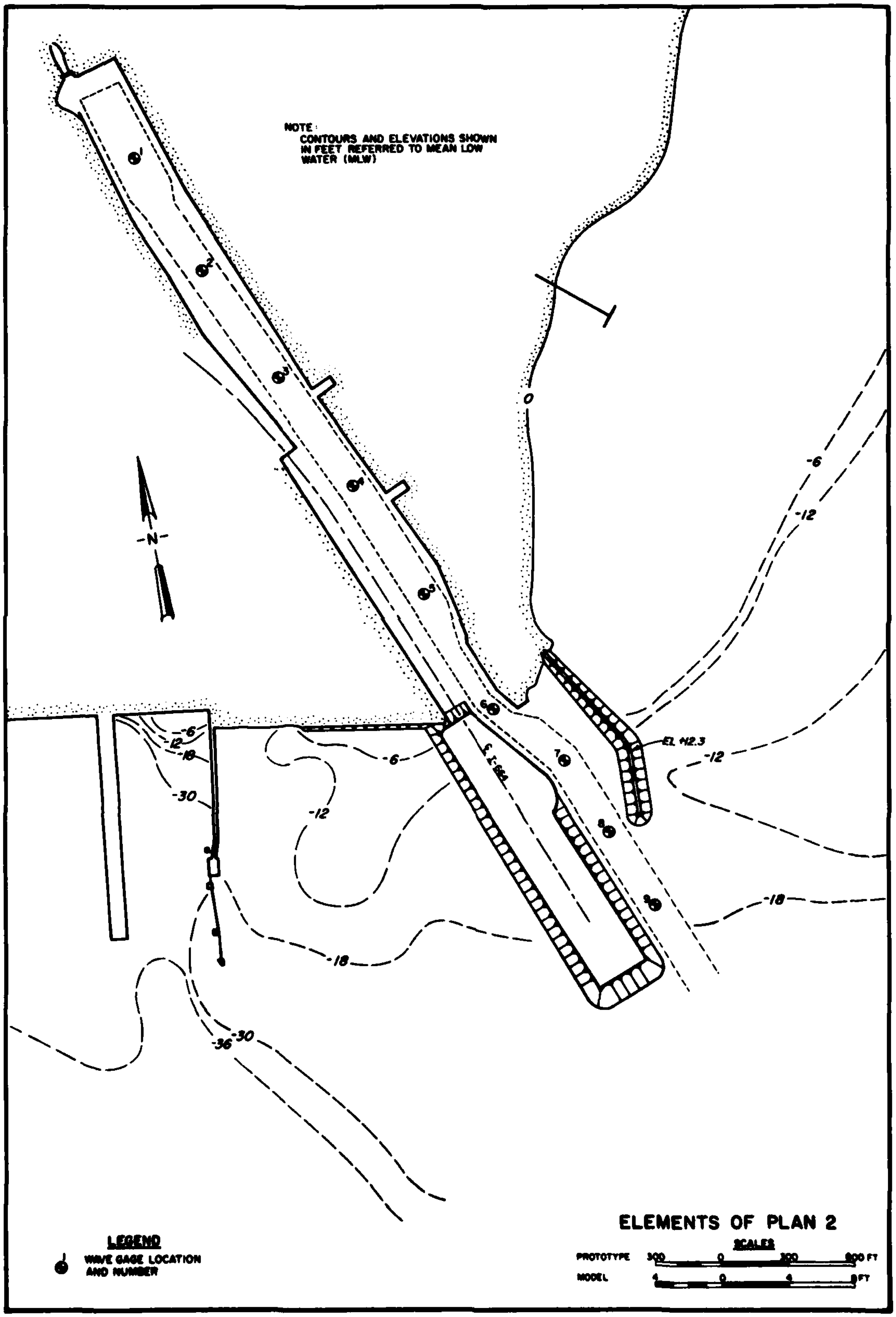

NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
N
0
-6
-12
-18
-30
-36
EL +12.3
ELEMENTS OF PLAN I
SCALES
PROTOTYPE 300 0 300 600 FT
MODEL 4 0 4 8 FT
LEGEND
WAVE GAGE LOCATION AND NUMBER

PLATE 2



ELEMENTS OF PLAN 2

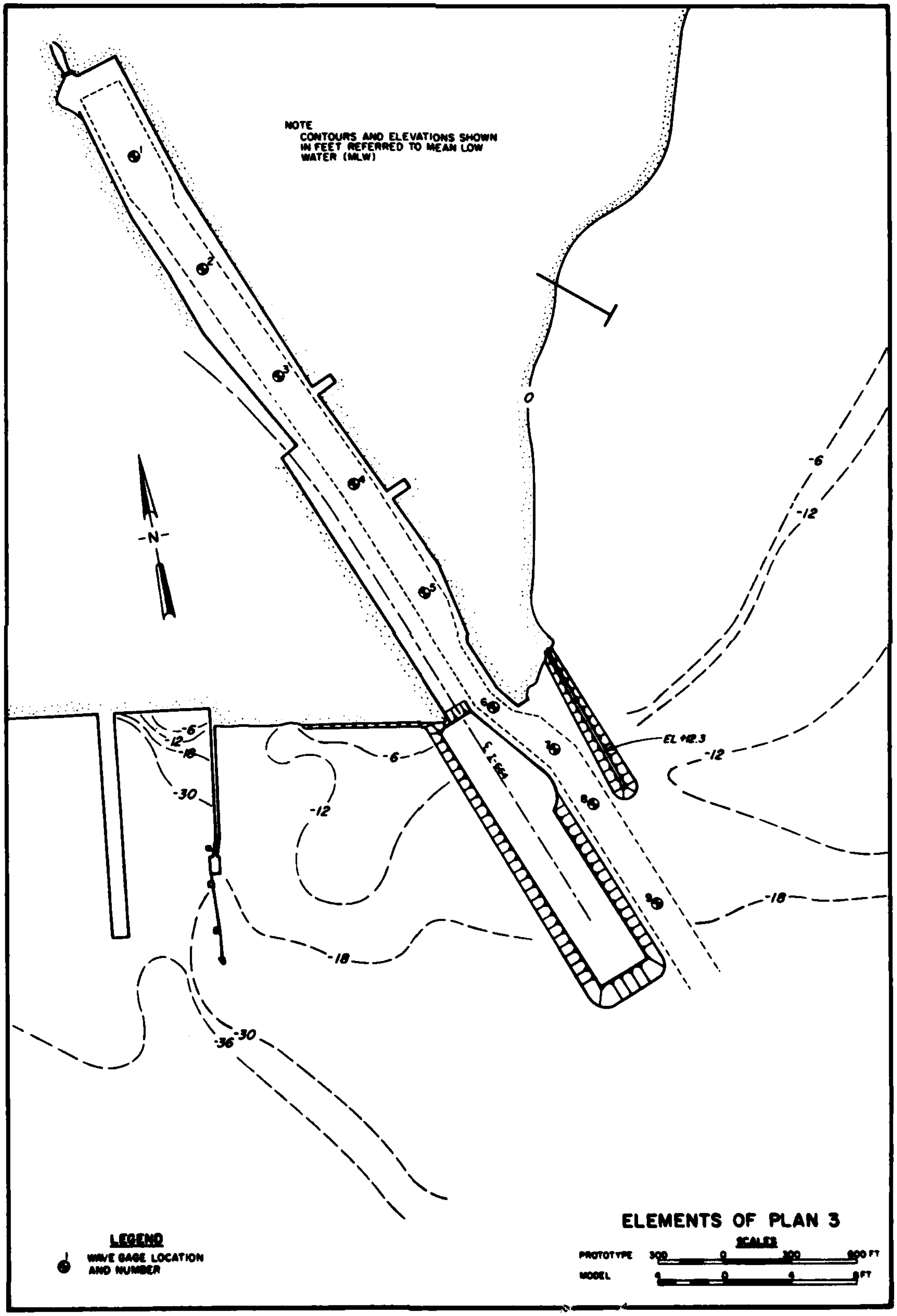
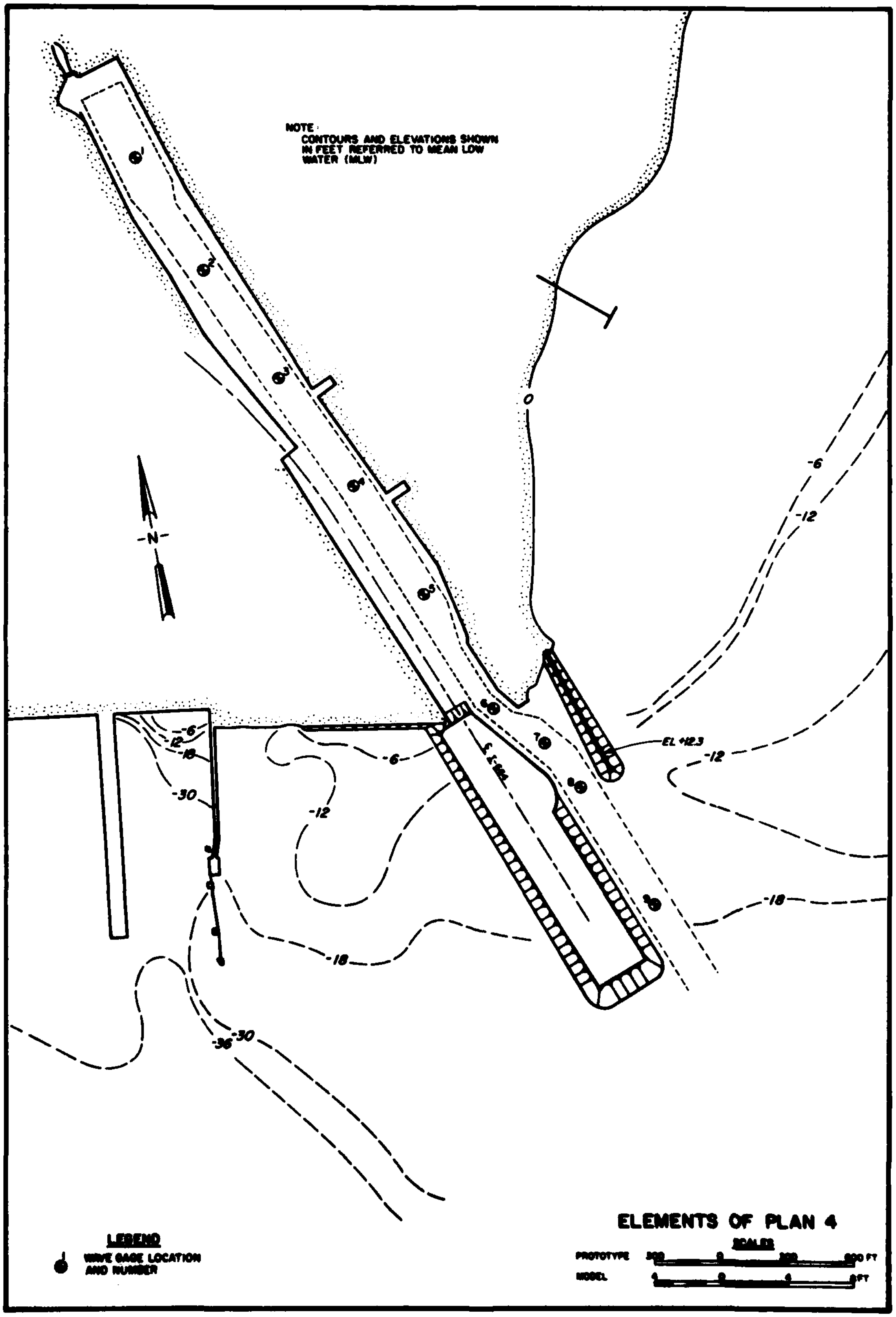

NOTE
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
N
0
-6
-12
-18
-30
-36
EL +12.3
LEGEND
WAVE GAGE LOCATION AND NUMBER
ELEMENTS OF PLAN 3
SCALES
PROTOTYPE 300 0 300 600 FT
MODEL 4 0 4 8 FT

PLATE 4



ELEMENTS OF PLAN 4

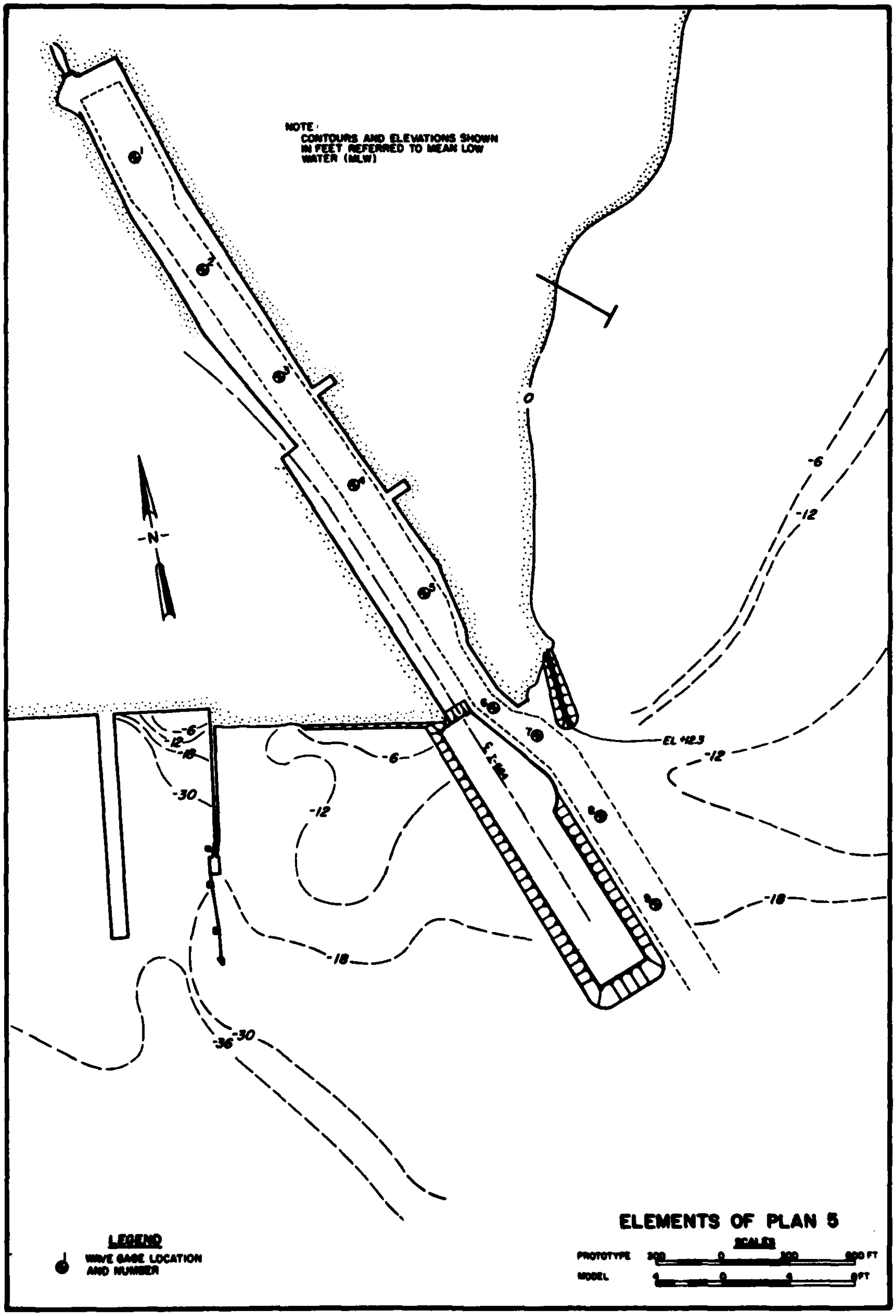

NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
N
0
-6
-12
-18
-30
-36
EL +12.3
LEGEND
WAVE GAGE LOCATION AND NUMBER
ELEMENTS OF PLAN 5
SCALES
PROTOTYPE
MODEL
FT

PLATE 6

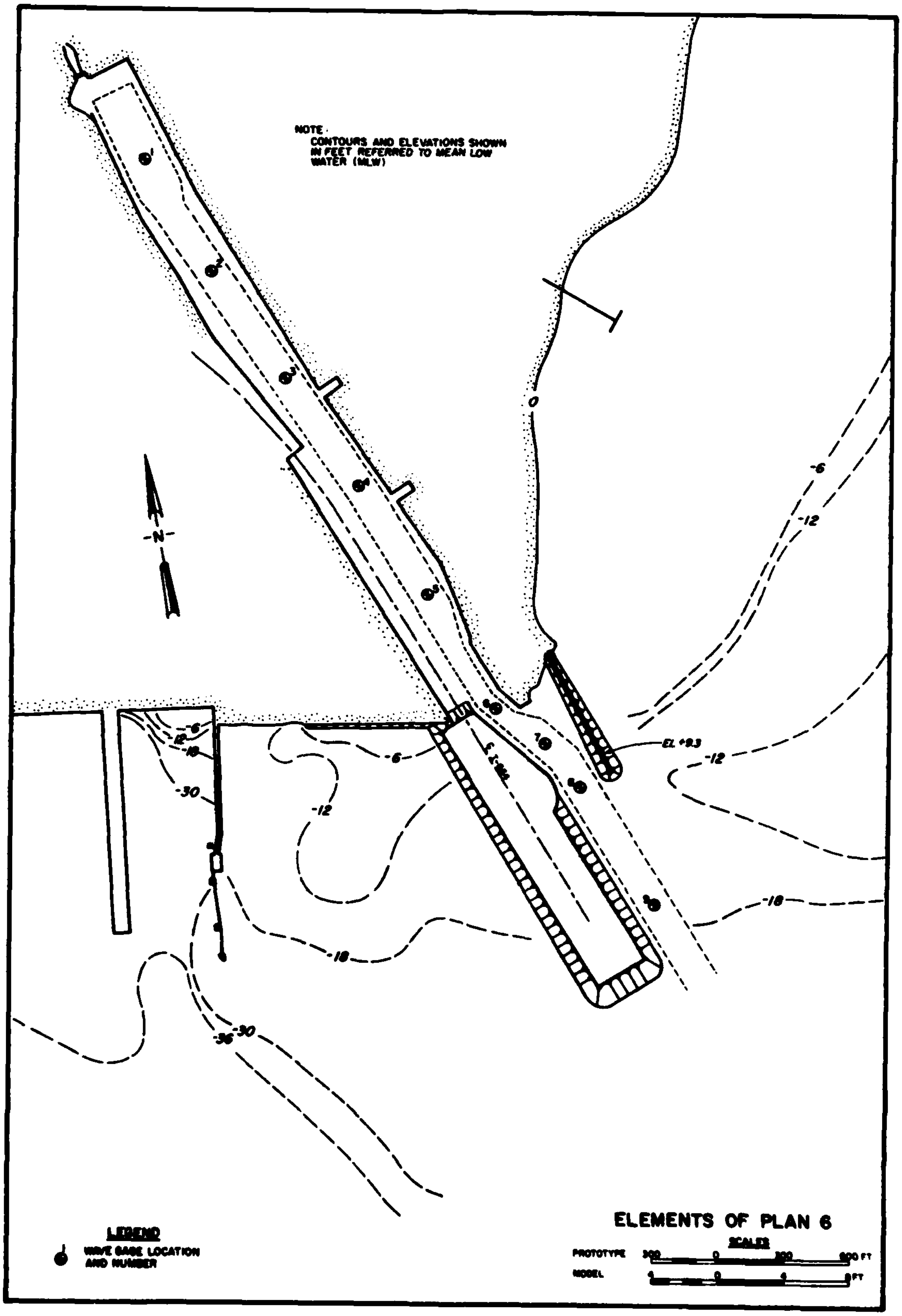

NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
N
EL +9.3
LEGEND
WAVE GAGE LOCATION AND NUMBER
ELEMENTS OF PLAN 6
SCALES
PROTOTYPE 300 0 300 600 FT
MODEL

PLATE 7

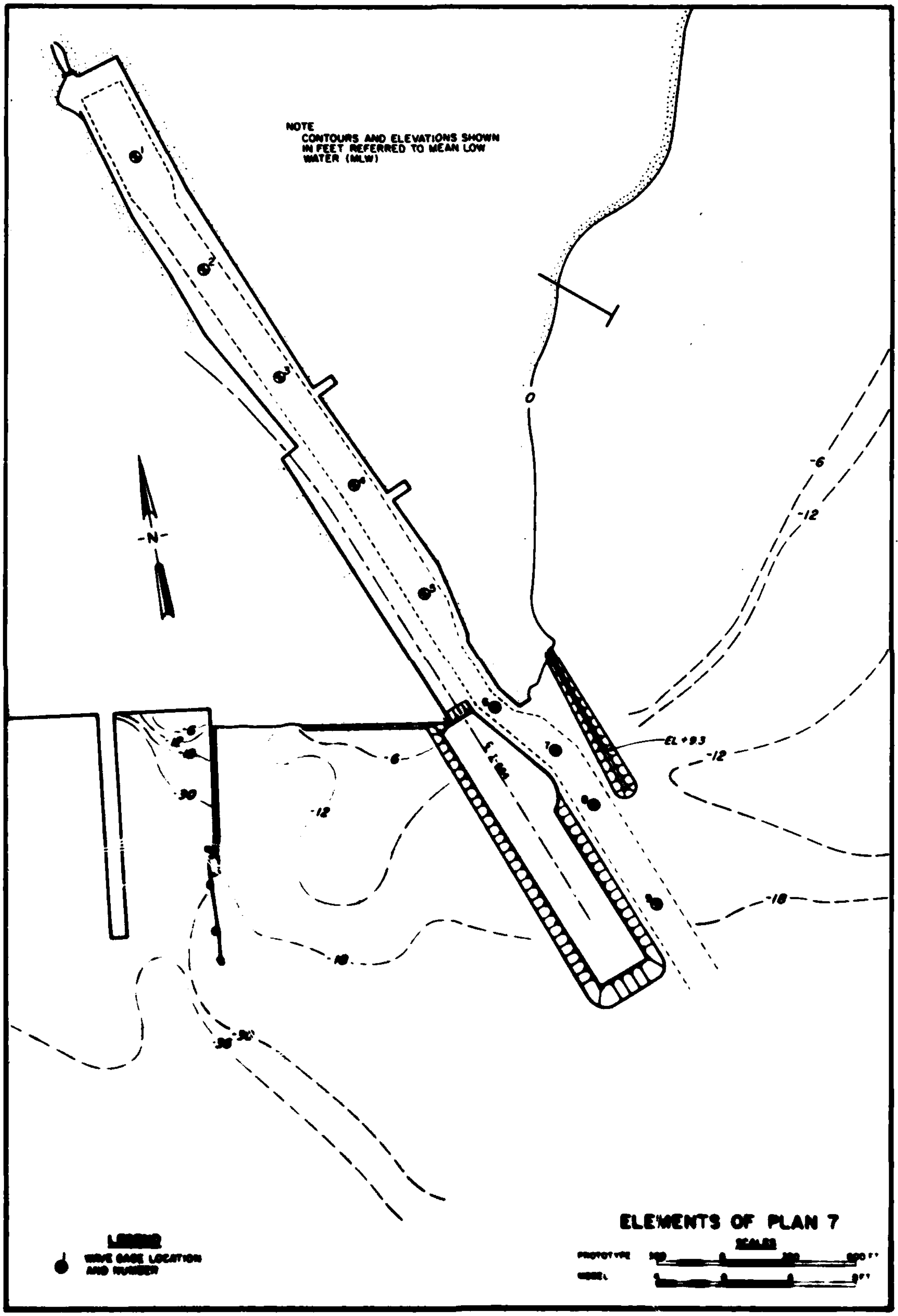
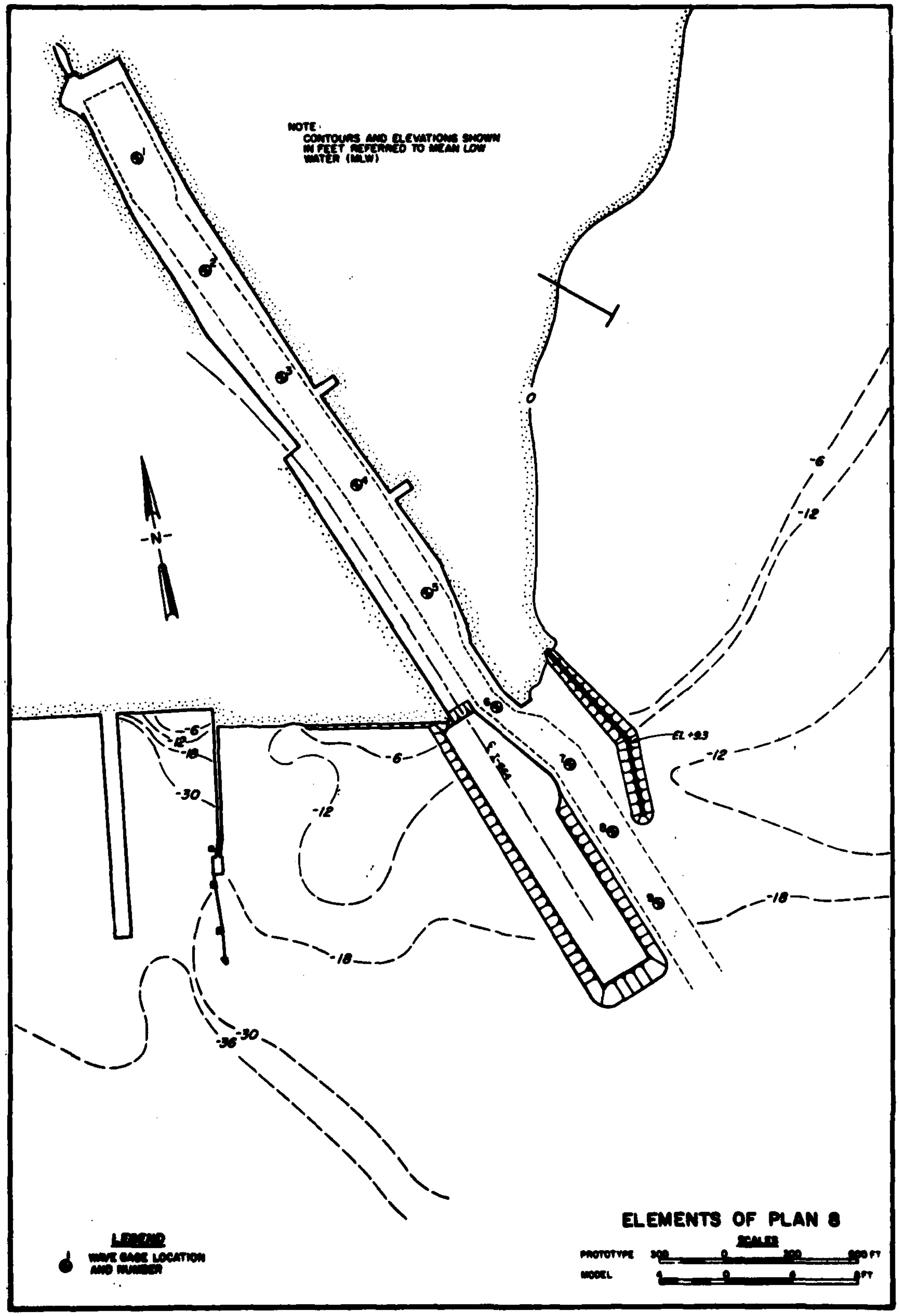

NOTE
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
N
0
-6
-12
-18
-30
EL +9.3
ELEMENTS OF PLAN 7
LEGEND
WAVE GAGE LOCATION AND NUMBER

PLATE 8



ELEMENTS OF PLAN 8

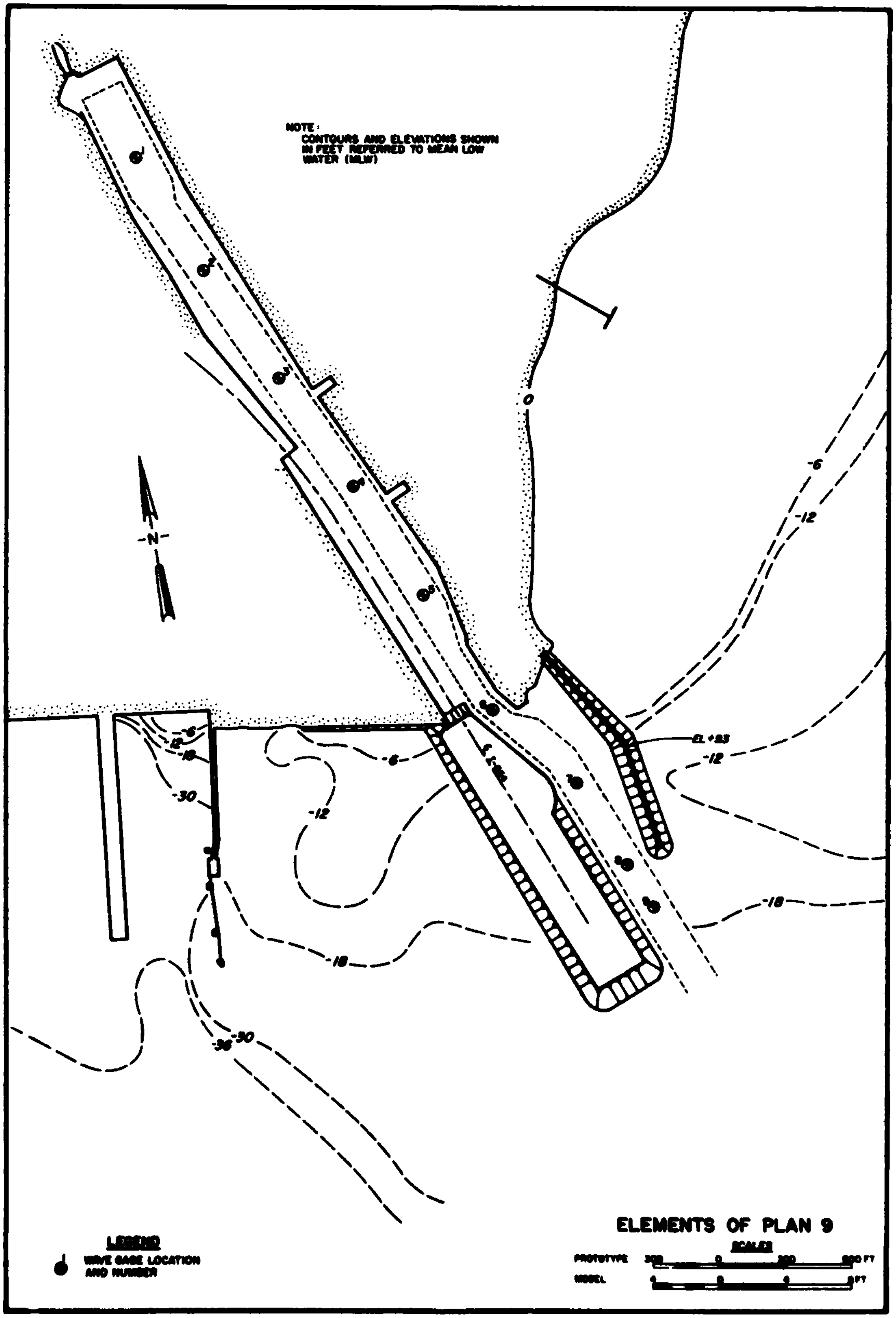

NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
N
0
-6
-12
-18
-30
-36
EL +23
ELEMENTS OF PLAN 9
SCALES
PROTOTYPE
MODEL
LEGEND
WAVE GAGE LOCATION AND NUMBER

PLATE 10

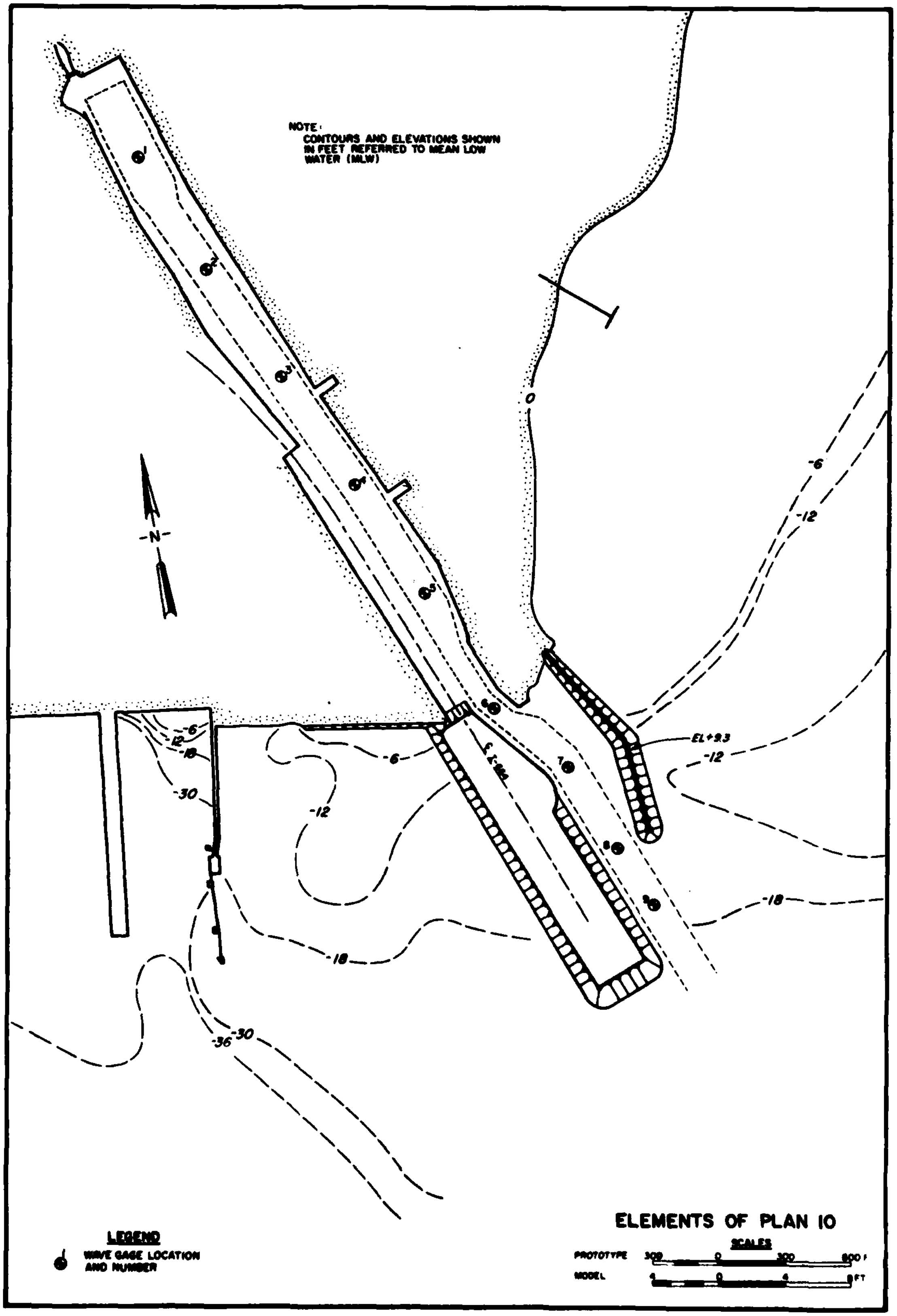
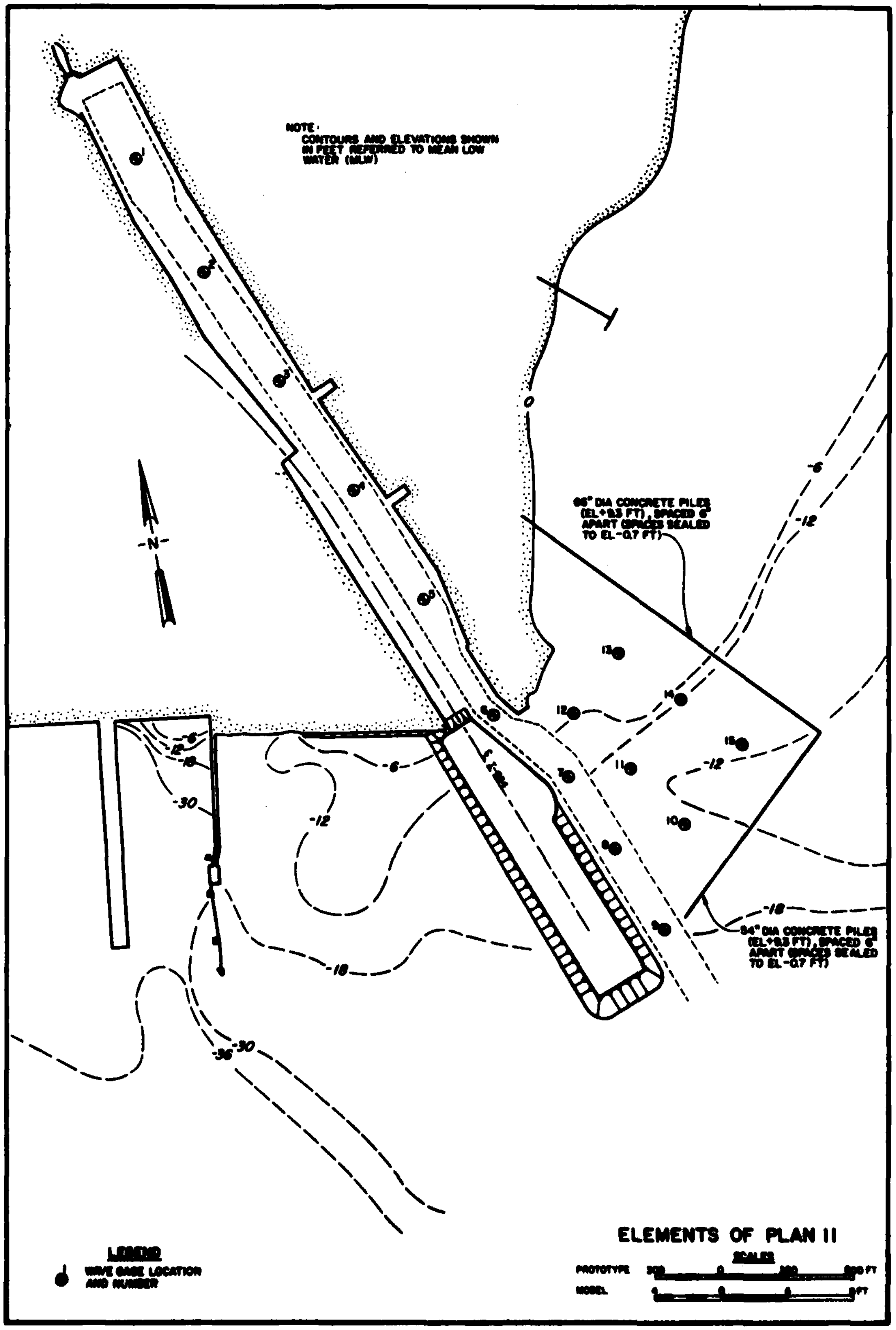

NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
N
0
-6
-12
-18
-30
-36
EL+9.3
LEGEND
WAVE GAGE LOCATION AND NUMBER
ELEMENTS OF PLAN 10
SCALES
PROTOTYPE
MODEL

PLATE 11



ELEMENTS OF PLAN II

PLATE 12

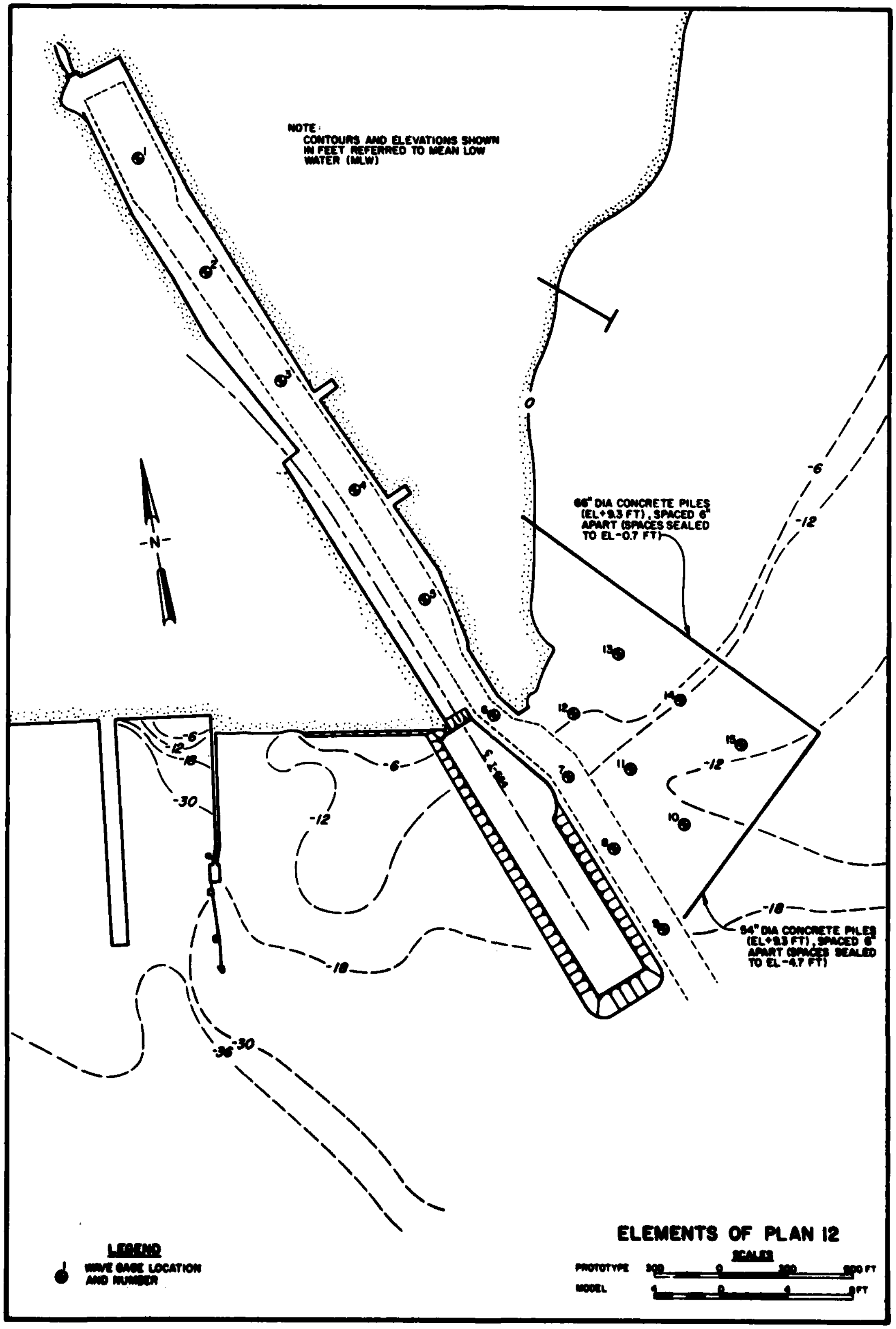

NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
N
66" DIA CONCRETE PILES (EL+9.3 FT), SPACED 6" APART (SPACES SEALED TO EL-0.7 FT)
54" DIA CONCRETE PILES (EL+9.3 FT), SPACED 6" APART (SPACES SEALED TO EL-4.7 FT)
LEGEND
WAVE GAGE LOCATION AND NUMBER
ELEMENTS OF PLAN 12
SCALES
PROTOTYPE 300 0 300 600 FT
MODEL 4 0 4 8 FT

PLATE 13

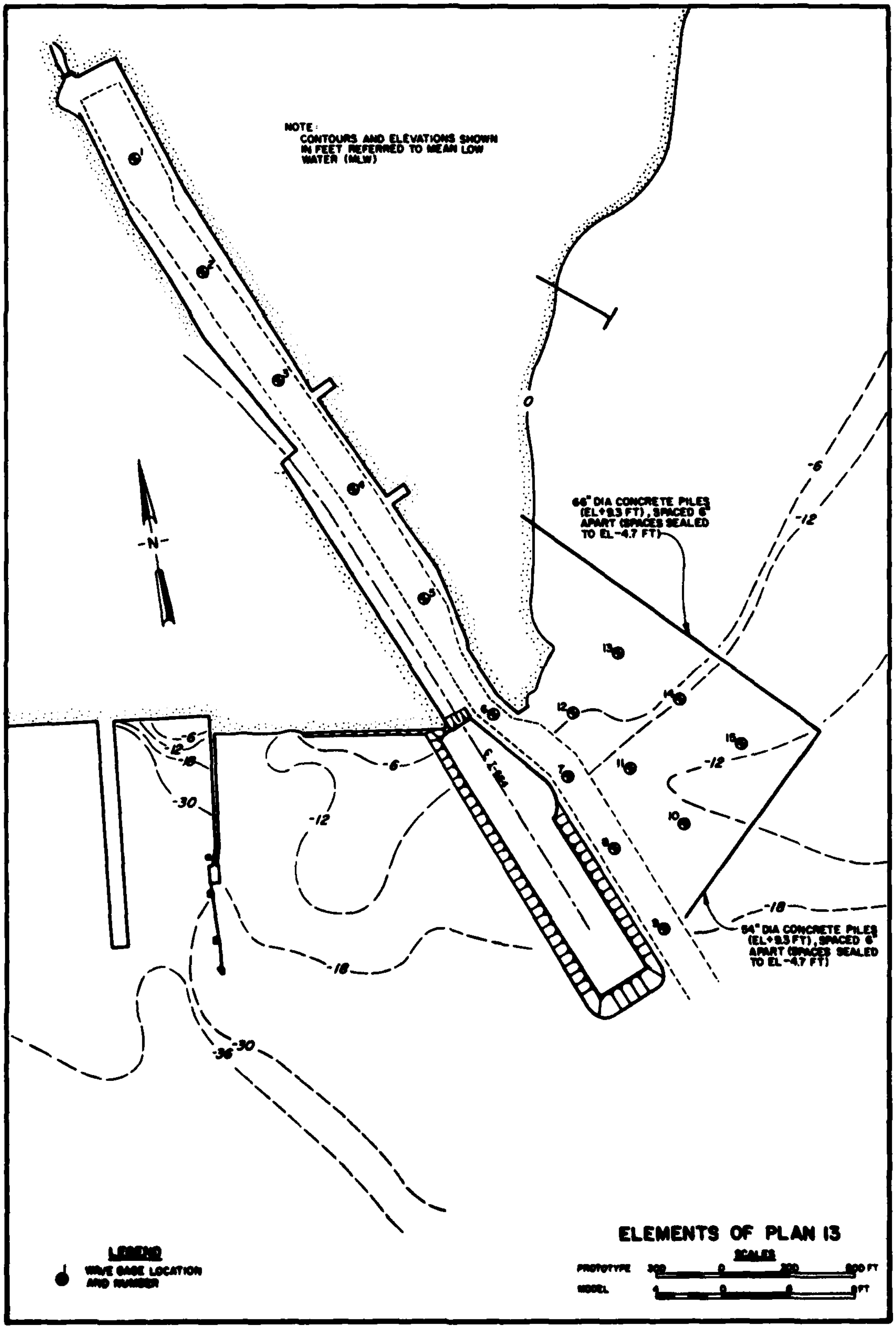

NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
N
66" DIA CONCRETE PILES (EL+9.3 FT), SPACED 6" APART (SPACES SEALED TO EL-4.7 FT)
54" DIA CONCRETE PILES (EL+9.3 FT), SPACED 6" APART (SPACES SEALED TO EL-4.7 FT)
ELEMENTS OF PLAN 13
LEGEND
WAVE GAGE LOCATION AND NUMBER
SCALES
PROTOTYPE
MODEL

PLATE 14

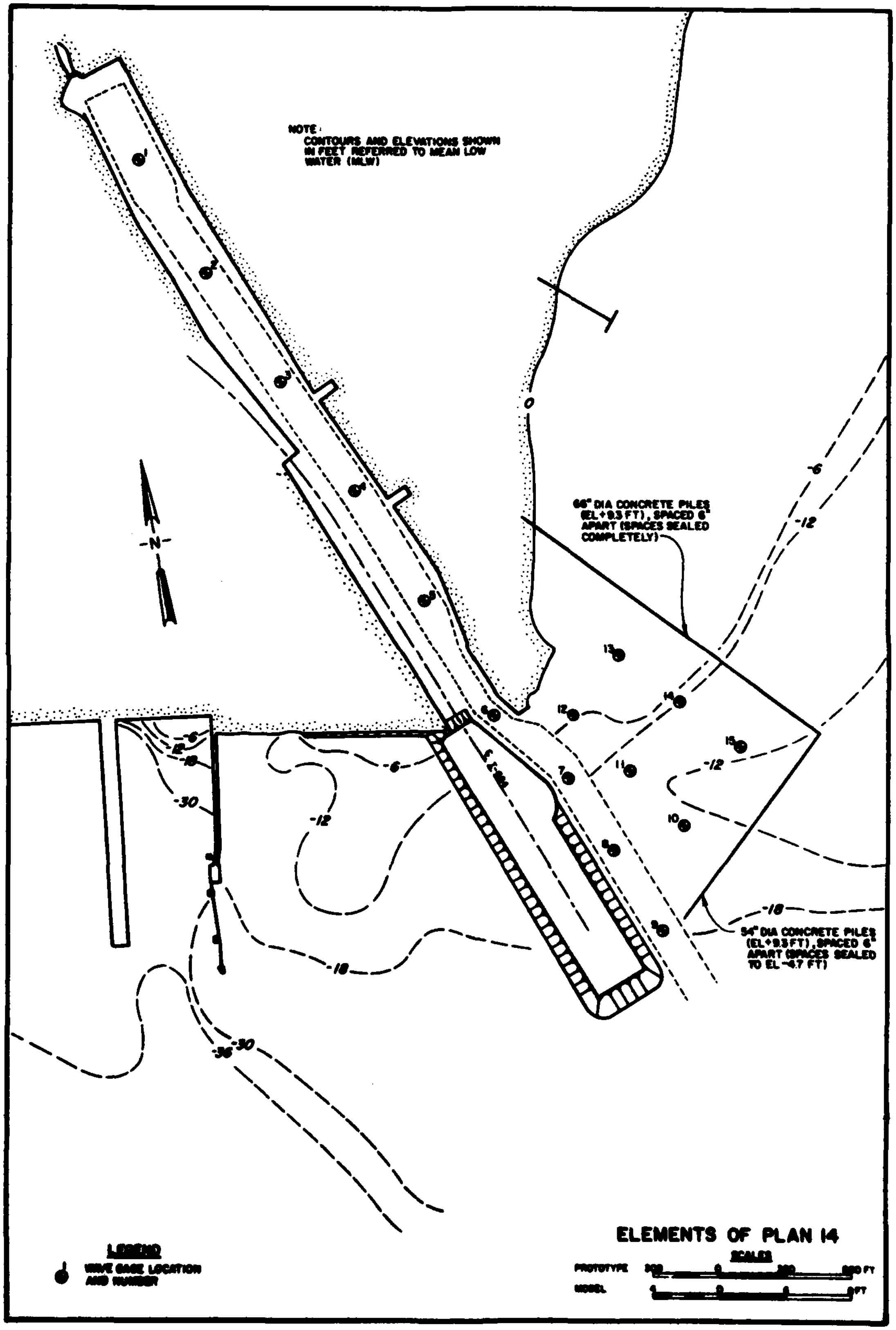

NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
N
0
-6
-12
-18
-30
-36
66" DIA CONCRETE PILES (EL+9.3 FT), SPACED 6" APART (SPACES SEALED COMPLETELY)
54" DIA CONCRETE PILES (EL+9.3 FT), SPACED 6" APART (SPACES SEALED TO EL -4.7 FT)
ELEMENTS OF PLAN 14
PROTOTYPE
FT
MODEL
FT

PLATE 15

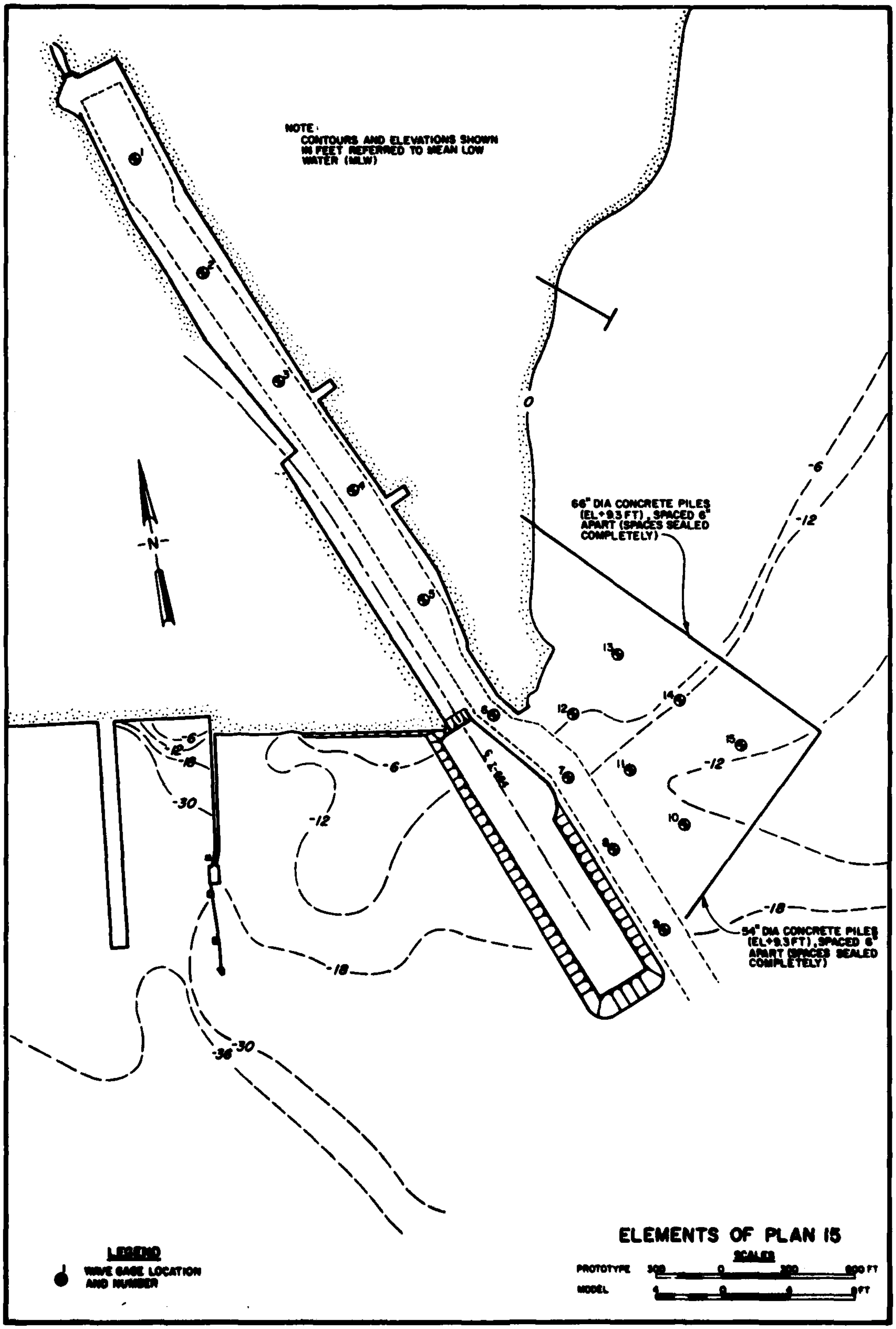
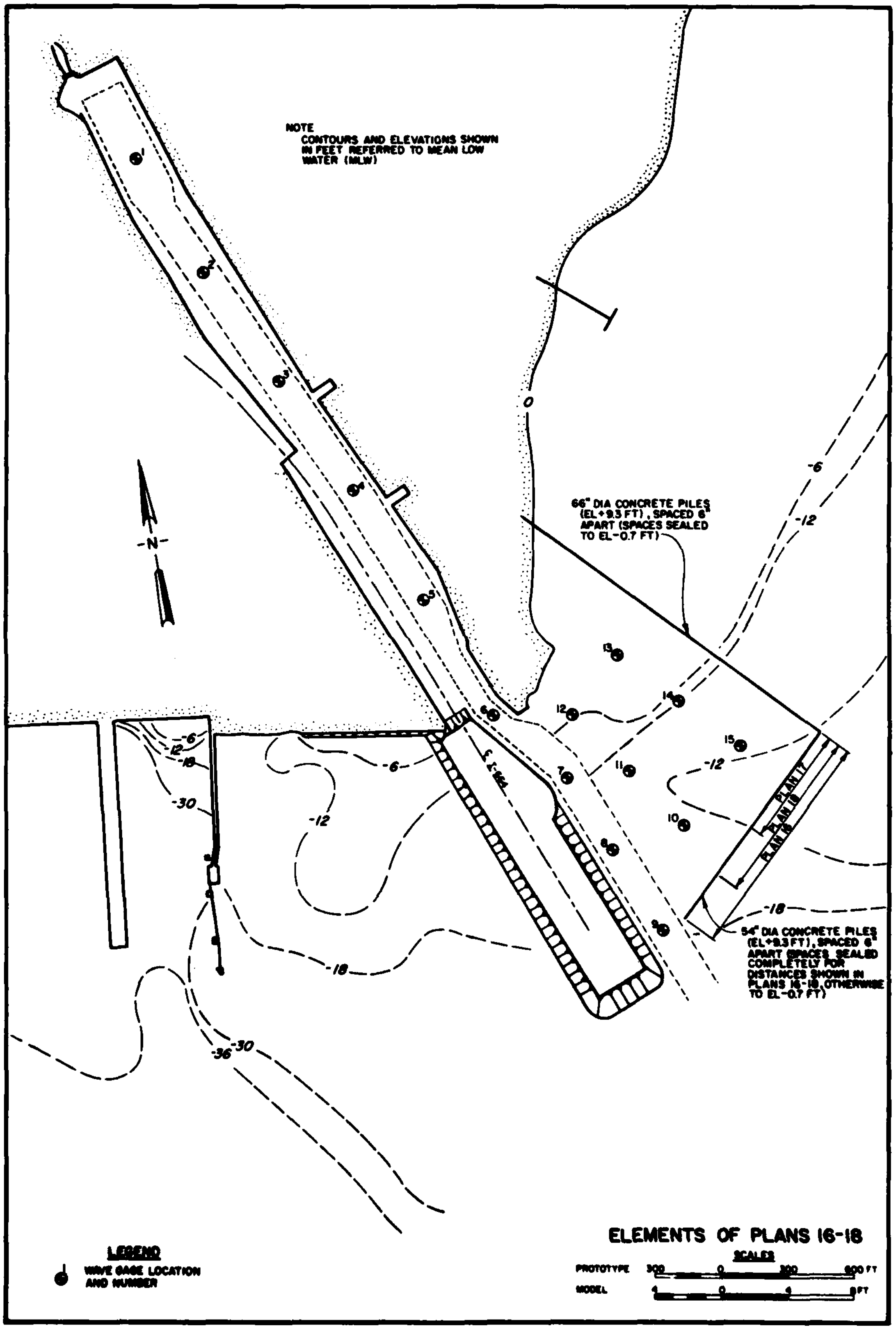

NOTE:
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOW WATER (MLW)
N
0
-6
-12
-18
-30
-36
66" DIA CONCRETE PILES (EL+9.3 FT), SPACED 6" APART (SPACES SEALED COMPLETELY)
54" DIA CONCRETE PILES (EL+9.3 FT), SPACED 6" APART (SPACES SEALED COMPLETELY)
LEGEND
WAVE GAGE LOCATION AND NUMBER
ELEMENTS OF PLAN 15
SCALES
PROTOTYPE 300 0 300 600 FT
MODEL 6 0 6 12 FT

PLATE 16



ELEMENTS OF PLANS 16-18

PLATE 17